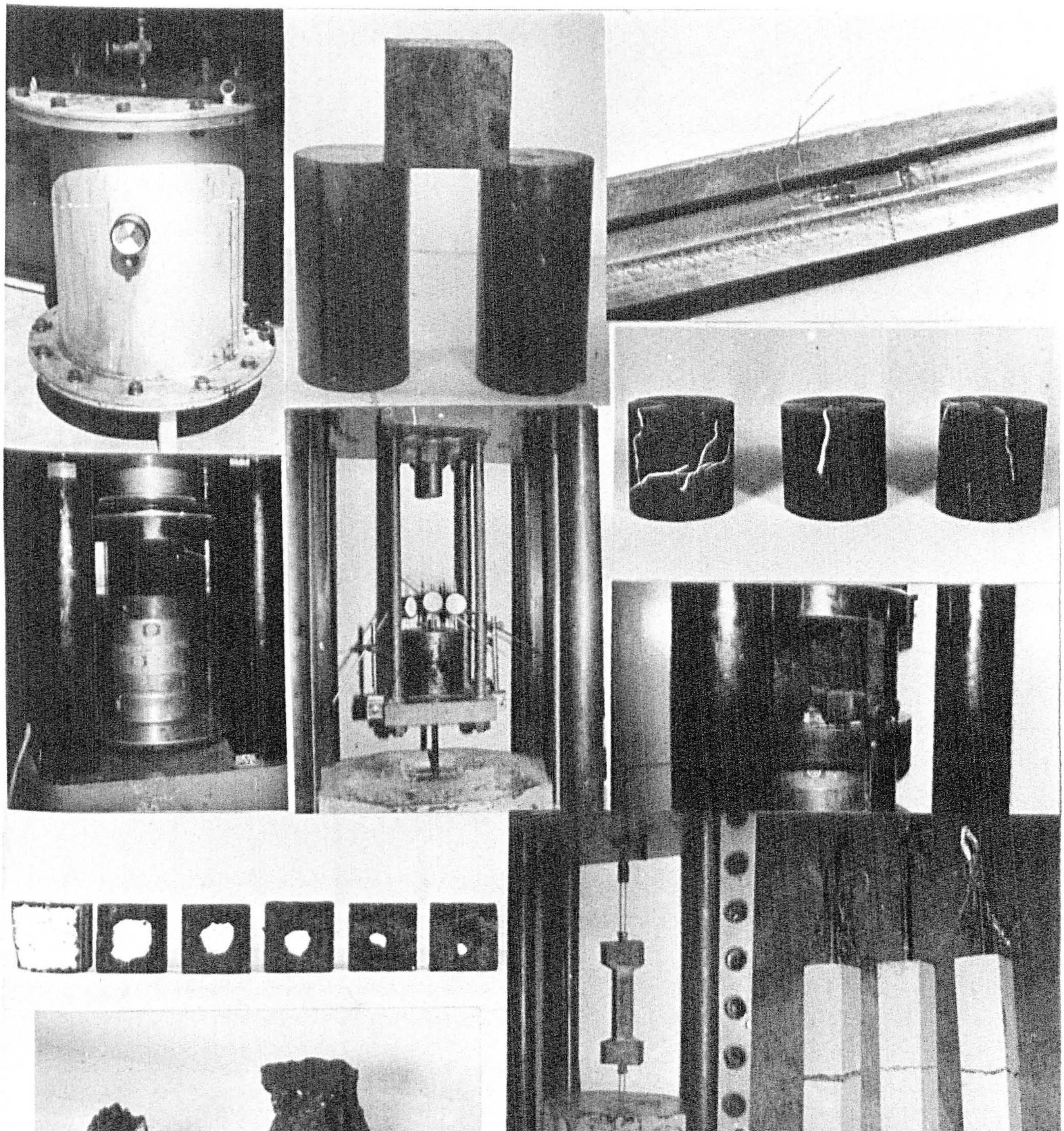
PROPERTIES OF OIL SATURATED CONCRETE

by

FARIS ISMAIL FAIYADH
B.Sc.(Iraq), M.Eng. (Sheffield)

Thesis submitted to the
University of Sheffield
for the Degree of
Doctor of Philosophy
in the Faculty of Engineering.

Department of Civil and Structural Engineering
May, 1980.


PLEASE CLEAN PRESS AFTER USE

To my parents

With

Lots of Love and Respect.

SUMMARY

Large concrete structures are being built for the production, storage and transportation of oil. The effects of crude oil on the properties of concrete are not well known and little data is available in the published literature.

The present investigation is divided into three parts:

1) A study of the mechanical properties of oil saturated hardened cement paste, mortar and concrete.
2) A study of the effect of absorbed oil on the average bond stress, and the bond stress distribution between reinforcing bar and concrete.
3) A study of the changes in the dynamic modulus and logarithmic decrement of concrete soaked in oil for different periods.

All the specimens were oven dried to increase the permeability of concrete to oil and to increase the degree of oil saturation. The results were compared with those of similar specimens soaked in water or sealed in polythene bags as control specimens.

The mechanical properties of HCPs, mortar, and concrete studied here were adversely affected by oil saturation. The compressive strength and elastic modulus were reduced by amounts depending on the amount of oil absorbed. The tensile strength was also reduced due to soaking in oil but the reduction was less than that of the companion specimens sealed as a reference. It appears that the oil mainly affects the aggregate-cement bond strength.

The average bond strength between steel and concrete, for both plain and deformed bars, decreases with an increase in the amount of absorbed oil. The reduction for the plain bar was between 1.8-2.3 times greater than that for the deformed bar. The study of the bond stress distribution showed that at the maximum applied load, the local bond stress at the embedded end of the oil saturated specimens was about six times greater than that at the loaded end.

The dynamic modulus of elasticity increased by 12-14% after soaking the concrete in oil for 580 days compared to the oven dried specimens. The increase was about 37% less than that of similar water soaked specimens. The studies of logarithmic decrement indicate that the loss of moisture is the most important factor influencing damping capacity.

ACKNOWLEDGEMENTS.

The author wishes to thank Professor Bond and the Department of Civil and Structural Engineering of the University of Sheffield for providing the facilities for conducting this research.

His gratitude goes to Dr. A.J. Watson,for his constant encouragement, criticisms and supervision throughout the period of the research.

Many thanks go also to the University of Technology, Baghdad, Iraq, for financial support.

Thanks are due to Shell Oil (U.K.) Limited for providing the crude oil.

The author also acknowledges the assistance of Mr. R. Newman and all technical staff in the concrete, rock, and light structures laboratories. Acknowledgements are due to Mr. Wright in the Department of Chemistry for analysing the crude oil samples.

CONTENTS

		Page No.
	Summary.	i.
	Acknowledgements.	iii.
	Contents.	iv.
	List of Figures.	ix.
	List of Tables.	xvi.
	List of Plates.	xix.
	Abbreviations.	xx.
CHAPTER 1.	INTRODUCTION.	
1.1	General Introduction.	1.
1.2	Effect of Oil on Concrete.	2.
1.3	Purpose of the Present Research.	5.
1.4	Scope of the Investigation.	6.
CHAPTER 2.	LITERATURE SURVEY.	
2.1	The Structure of Cement Paste and Concrete.	8.
2.2	The Strength of Concrete.	9.
2.2.1	The Cement Paste Strength.	10.
2.2.2	The Aggregate-Cement Bond Strength.	11.
2.3	Internal Microcracking in Concrete.	12.
2.4	Failure of Concrete Test Specimens.	15.
2.4.1	Strength in Compression.	15.
2.4.2	Strength in Tension.	17.
2.4.2.1	The splitting Tensile Test.	18.
2.4.2.2	The flexural Test.	19.
2.4.2.3	The Uniaxial Tensile Test.	20.
2.5	The Effect of Drying on Strength of Concrete Specimens.	22.
2.6	Volume Changes of Concrete According to Moisture Movement.	24.
2.6.1	Factors Effecting the Drying Shrinkage of Concrete.	25.
2.7	Dynamic and Damping Capacity of Concrete.	27.
2.7.1	Determination of Modulus of Elasticity from Dynamic Tests.	28.
2.7.2	Determination of the Damping Capacity of Concrete.	30.
2.8	Bond Strength.	32.
2.8.1	Nature of Bond.	32.
2.8.2	Bond Tests.	35.
2.8.3	Bond Stress Distribution and the Techniques Used in its Measurements.	36.

Page No.

CHAPTER 3. EXPERIMENTAL PROGRAMME AND MATERIALS.

3.1 Introduction. 41.
3.2 Experimental Programme. 41.
3.3 Materials. 42.
3.3.1 Cement. 42.
3.3.2 Aggregates. 42.
3.3.3 Steel. 45.
3.3.4 Crude Oil. 46.
3.4 Preparation of Cement Paste and Mortar Specimens. 47.
3.4.1 Casting. 47.
3.4.2 Sizes and Shapes of the HCP and Mortar Specimens. 48.
3.5 Preparation of Concrete Specimens. 48.
3.5.1 Concrete Mixes. 48.
3.5.2 Casting of Concrete Specimens. 49.
3.5.3 Shapes and Sizes of Concrete Specimens. 50.
3.6 Curing and Storage. 51.

CHAPTER 4. SOME MECHANICAL PROPERTIES OF OIL SATURATED CEMENT PASTE, MORTAR, AND CONCRETE.

4.1 Introduction. 53.
4.2 Experimental Programme and Procedure. 53.
4.2.1 Cement Pastes and Mortars. 53.
4.2.2. Plain Concrete. 54.
4.2.2.1 The Compressive Strength. 54.
4.2.2.2 The Tensile Strength. 54.
4.2.2.3 The Static Modulus of Elasticity. 56.
4.2.2.4 Ultrasonic-Pulse Velocity Test. 57.
4.2.2.5 Volume Change Measurements. 57.
4.3 Test Results for Hardened Cemente Pastes and Mortars. 58.
4.3.1 Introduction. 58.
4.3.2 Effect of Oil Saturation on the Cube Crushing Strength of HCPs and Mortars. 59.
4.3.2.1 Effect of Oil Saturation on the Cube Crushing Strength at Various Ages. 59.
4.3.2.2 Effect of Oil Saturation on the Cube Crushing Strength at various w/c Ratios. 62.
4.3.3 Static Modulus of Elasticity of HCPs and Mortars. 63.
4.3.3.1 Stress-Strain Curves. 63.
4.3.3.2 Effect of Crude Oil Saturation on the Elastic Modulus and Prism Strength of HCPs and Mortars. 63.
4.3.4 Effect of Oil Saturation on the Direct Tensile Strength of HCPs and Mortars. 64.
4.4 Test Results for Plain Concrete. 67.
4.4.1 Effect of Oil Saturation on the Compressive Strength of Concrete. 67.

Page No.

4.4.1.1 Effect of Prolonged Storage in Crude Oil on the Cube Crushing Strength of Concrete. 67.
4.4.1.2 The Effect of Oil Saturation on the Cube Crushing Strength of Concrete at Various w/c Ratios. 71.
4.4.1.3 Effect of Oil Saturation on the Cylinder and Prism Compressive Strength. 73.
4.4.2 Effect of Oil Saturation on the Splitting Tensile Strength of Concrete. 74.
4.4.3 Effect of Oil Saturation on the Direct Tensile Strength of Concrete. 76.
4.4.4 The Effect of Oil Saturation on the Modulus of Rupture of Concrete. 78.
4.4.5 The Effect of Oil Saturation on the Elastic Modulus of Concrete. 80.
4.4.5.1 The Stress-Strain Curve. 80.
4.4.5.2 The Effect of Oil Saturation on the Elastic Modulus of Concrete at Various Ages. 81.
4.4.5.3 The Effect of Oil Saturation on the Elastic Modulus of Concrete at Various w/c Ratios. 86.
4.4.6 Volumetric Changes Under Compressive load. 87.
4.4.7 Poisson's Ratio of Oil Saturated Concrete. 87.
4.4.8 The Ultrasonic-Pulse Velocity of Concrete Cubes under Uniaxial Compressive Load. 88.
4.4.9 Moisture Movement and Volume Changes. 89.
4.4.10 The Effect of Total Porosity on the Compressive Strength of Oil Saturated Concrete. 91.
4.4.10.1 Determination of Total Porosity. 91.
4.4.10.2 Variation of the Compressive Strength of Concrete With the Total Porosity. 93.
4.4.11 Oil Absorption Characteristics of HCPs and Mortars. 95.
4.4.11.1 Introduction. 95.
4.4.11.2 Absorption Characteristics of HCPs and Mortars. 95.
4.4.12 Absorption Characteristics of Plain Concrete. 98.
4.4.12.1 Variation of Oil Absorption with Time. 98.
4.4.12.2 The Variation of Depth of Oil Penetration with Time. 101.
4.4.12.3 Variation of Oil Absorption with w/c Ratio and Total Porosity. 102.
4.5 Discussion of the Test Results. 105.
4.5.1 Effect of Drying on the Strength of Concrete. 105
4.5.2 The Effect of Oil Saturation on the Cube Crushing Strength of HCP, Mortar, and Concrete. 107.
4.5.3 The Effect of Absorbed Oil on the Cylinder and Prism Compressive Strength of Concrete. 114.
4.5.4 The Effect of Oil Saturation on the Tensile Strength of Concrete. 116.
4.5.5 The Effect of Water and Oil Saturation on the Volume Changes. 119.

Page No.

4.5.6 Effect of Oil Saturation on the Elastic Modulus of HCPs, Mortars, and Concrete. 123.
4.5.7 Relationship Between Elastic Modulus and Compressive Strength of Concrete. 125.

CHAPTER 5. BOND CHARACTERISTICS OF OIL SATURATED CONCRETE.

5.1 Introduction. 128.
5.2 Scope of the Tests. 128.
5.3 Experimental Programme and Test Procedure. 128.
5.3.1 Pull-Out Test. 128.
5.3.1.1 Steel Reinforcement. 128.
5.3.1.2 Test Programme. 129.
5.3.1.3 Specimen Preparation. 129.
5.3.1.4 Testing Procedure. 130.
5.3.2 Modified Pull-Out Test. 131.
5.3.2.1 Test Programme. 131.
5.3.2.2 Instrumented Bar. 131.
5.3.2.3 Preparation of Test Specimens. 132.
5.3.2.4 Testing Procedure. 133.
5.4 Test Results and Discussion. 134.
5.4.1 Pull-Out Test. 134.
5.4.1.1 Behaviour at Failure. 134.
5.4.1.2 The Effect of Absorbed Oil on the Average Bond Strength. 135.
5.4.1.3 Load-Slip Relationship. 138.
5.4.1.4 Mechanism of Bond Failure in Pull-Out Test. 141.
5.4.2 Modified Pull-Out Test. 143.
5.4.2.1 General Behaviour. 143.
5.4.2.2 Load-Slip Relationship. 146.
5.4.2.3 The Effect of Storage Conditions on the Load-Transfer and Bond Stress Distribution. 147.

CHAPTER 6. DYNAMIC PROPERTIES OF OIL SATURATED CONCRETE.

6.1 Introduction. 153.
6.2 Experimental Programme. 153.
6.3 Apparatus. 153.
6.4 Testing Procedure. 154.
6.5 Test Results and Discussion. 156.
6.5.1 Dynamic Modulus of Elasticity. 156.
6.5.1.1 Effect of Drying on the Dynamic Modulus of Elasticity. 156.
6.5.1.2 The Effect of Age on the Dynamic Modulus of Elasticity of Concrete. 158.
6.5.1.3 Effect of Absorbed Oil on the Dynamic Modulus of Elasticity. 162.
6.5.1.4 The Effect of w/c Ratio on the Dynamic Modulus of Concrete. 163.
6.5.1.5 The Relation Between the Resonant Frequency and Storage Conditions. 164.
6.5.1.6 The Relationship Between the Static and Dynamic Modulus of Concrete. 167.
6.5.2 The Damping Capacity of Concrete. 170.

Page No.

6.5.2.1 Effect of w/c Ratio and the Maximum Size of Aggregate on the Damping Capacity of Concrete. 171.
6.5.2.2 The Effect of Age on the Damping Capacity of Concrete. 173.
6.5.2.3 The Effect of Water and Oil Penetration on the Damping Capacity of Dry Concrete. 175.
6.5.2.4 The Relation Between Logarithmic Decrement, Compressive Strength and Dynamic Modulus of Concrete. 176.

CHAPTER 7. CONCLUSIONS, LIMITATIONS OF THE PRESENT WORK, AND PROPOSALS FOR FUTURE WORK.

7.1 Conclusions. 178.
7.1.1 Mechanical Properties. 178.
7.1.1.1 Hardened Cement Paste and Mortar. 178.
7.1.1.2 Plain Concrete. 179.
7.1.2 Oil Absorption. 181.
7.1.3 Bond Strength. 182.
7.1.3.1 Pull-Out Test. 182.
7.1.3.2 Modified Pull-Out Test. 183.
7.1.4 Dynamic Modulus of Elasticity. 184.
7.1.5 Damping Capacity. 185.
7.2 Limitations of the Present Work. 186.
7.2.1 Materials. 186.
7.2.2 Mechanical Properties. 186.
7.2.3 Dynamic and Damping Properties. 186.
7.2.4 Bond Strength. 187.
7.3 Recommendations for Future Work. 187.

References. 189.

LIST OF FIGURES

Fig.No.	Title	Preceeding Page No.
1.1	The compressive strength of oil soaked concrete vs. Age.	4.
1.2	The tensile strength of oil soaked concrete vs. age.	4.
2.1	Typical stress-strain behaviour of concrete or mortar under compression.	15.
2.2	Stress distribution normal to the loaded plane for split test specimen.	18.
2.3	Details for the pull-out test specimen.	33.
2.4	Deformation of concrete around reinforcing bars.	34.
3.1	Grading curves for 10 and 20 mm coarse aggregates.	45.
3.2	Grading curves for fine aggregate.	45.
3.3	Short term stress-strain curve for the bars used in the pull-out specimens.	46.
3.4	Details of tensile specimen and end cap.	50.
4.1	Effect of storage conditions on the compressive strength of pastes at various ages and w/c ratios.	60.
4.2	Effect of storage conditions on the compressive strength of 1:2 mortars at various ages and w/c ratios.	60.
4.3	Effect of storage conditions on the compressive strength of 1:3 mortars at various ages and w/c ratios.	60.
4.4	Variation of compressive strentgh with w/c ratio and storage conditions.	60.
4.5	Stress-strain relationship showing the effect of storage conditions and w/c ratio on hardened cement paste specimens (at 300 days).	63.
4.6	Stress-strain relationship showing the effect of storage conditions on mortar specimens (at 300 days).	63.
4.7	Stress-strain relationship showing the effect of storage conditions on mortar specimens (at 300 days).	63.
4.8	Effect of storage conditions and w/c ratio on the elastic modulus of HCPs and mortars.	64.
4.9	The effect of storage conditions on the direct tensile strength of HCPs and mortars of various w/c ratio (at 150 days soaking period).	64.

Fig.No.	Title	Preceeding Page No.
4.10	The effect of curing condition on the cube crushing strength of concrete.	67.
4.11	The effect of curing condition on the cube crushing strength of concrete.	67.
4.12	The effect of curing condition on the cube crushing strength of concrete.	67.
4.13	The effect of storage condition and w/c ratio on the cube crushing strength of concrete.	72.
4.14	The effect of curing condition on the compressive strength of concrete cylinder.	73.
4.15	The effect of curing condition on the compressive strength of concrete cylinder.	73.
4.16	The effect of curing condition on the compressive strength of concrete cylinder.	73.
4.17	The effect of curing condition on the compressive strength of concrete prism.	73.
4.18	The effect of curing condition on the compressive strength of concrete prism.	73.
4.19	The effect of curing condition on the compressive strength of concrete prism.	73.
4.20	The effect of storage condition on the tensile splitting strength of concrete of various mix proportions.	75.
4.21	The effect of oil saturation on the direct tensile strength of concrete at various ages and w/c ratios.	76.
4.22	The effect of curing condition on the flexural strength of concrete at various ages.	79.
4.23	The effect of curing condition on the flexural strength of concrete at various ages.	79.
4.24	The effect of curing condition on stress-strain curve of concrete-Mix A (at 600 days).	80.
4.25	The effect of curing condition on the stress-strain curve of concrete-Mix B (at 600 days).	80.
4.26	Variation of the elastic modulus of concrete with period of storage in oil.	81.
4.27	Variation of the elastic modulus of concrete with the period of storage in oil.	81.
4.28	Effect of storage conditions and w/c ratio on the elastic modulus of concrete.	86.
4.29	Typical plots of volumetric changes under compressive load (at 600 days).	87.
4.30	Typical plots of volumetric changes under compressive load (at 600 days).	87.

Fig. No.	Title	Preceeding Page No.
4.31	Typical plots of Poisson's ratio changes of concrete (Mix A) under compressive load (at 600 days).	88.
4.32	Typical plots of Poisson's ratio changes of concrete (Mix B) under compressive load (at 600 days).	88.
4.33	The variation of pulse velocity with applied normal stress of concrete cube soaked at different curing condition for 600 days.	88.
4.34	The variation of pulse velocity with applied normal stress of concrete cube soaked at different curing condition for 600 days.	88.
4.35	Volume change of concrete vs. age.	90.
4.36	Volume change of concrete vs. age.	90.
4.37	Volume change of concrete vs. age.	90.
4.38	Volume change of concrete vs. age.	90.
4.39	Relationship between compressive strength and total porosity of concrete.	93.
4.40	Weight of absorption and water lost on drying to 105° C of HCPs.	95.
4.41	Volume of absorption and water lost on drying to 105° C of HCPs.	95.
4.42	Variation of absorption and water loss during drying for mortar specimens.	95.
4.43	Volume of absorption and water loss during drying for mortar specimens.	95.
4.44	Variation of absorption with total porosity of HCPs.	98.
4.45	Variation of absorption with total porosity of mortar.	98.
4.46	Variation of absorption with soaking period for concrete of 10 mm maximum size of aggregate.	98.
4.47	Variation of absorption with soaking period for concrete of 20 mm maximum size of aggregate.	98.
4.48	Average depth of penetration and area reached by oil for concrete cubes or prisms, 100x100 mm cross-section, vs. soaking period.	101.
4.49	Absorption after 600 days resoaking and water lost by drying at 105° C for concrete specimens.	102.
4.50	Volume of absorption after 600 days resoaking and water lost by drying at 105° C for concrete specimens.	102.

Fig.No.	Title	Preceeding Page No.
4.51	Variation of absorption with total porosity for concrete specimens.	104.
4.52	The relationship between the compressive strength of oil soaked concrete and oil absorption.	109.
4.53	Percentage reduction in strength due to oil saturation vs. wt. of absorbed oil.	109.
4.54	Reduction in strength of concrete due to oil saturation for 600 days vs. total porosity.	114.
4.55	Relationship between cube crushing strength and uniaxial compressive strength of concrete cylinders.	116.
4.56	Relationship between cube crushing strength and uniaxial compressive strength of concrete prisms.	116.
4.57	Relationship between modulus of rupture and cube crushing strength of concrete, Mix A.	118.
4.58	Relationship between modulus of rupture and cube crushing strength of concrete, Mix B.	118.
4.59	Relationship between splitting tensile strength and cube crushing strength of concrete.	119.
4.60	Variation of elastic modulus with the compressive strength of concrete.	126.
5.1	Details of pull-out testing arrangement.	130.
5.2	Details of the modified pull-out specimen.	131.
5.3	Steps of bar preparation and gauge locations for the modified pull-out test.	132.
5.4	Load-slip curves drawn automatically by the testing machine - Plain Bars.	134.
5.5	Load-slip curves drawn automatically by the testing machine - Deformed Bars.	134.
5.6	Average bond strength vs. age, plain bars.	135.
5.7	Average bond strength vs. age, high stress-high bond deformed bars.	135.
5.8	Applied load vs. free end slip for plain bar specimens tested at the age of 120 days.	138.
5.9	Applied load vs. free end slip for the plain bar specimens tested at the age of 300 days.	138.
5.10	Applied load vs. free end slip for the plain bar specimens tested at the age of 755 days.	138.
5.11	Applied load vs. free end slip for the deformed bar specimens tested at the age of 120 days.	138.

Fig.No.	Title	Preceeding Page No.
5.12	Applied load vs. free end slip for the deformed bar tested at the age of 320 days.	138.
5.13	Applied load vs. free end slip for the deformed bar tested at the age of 755 days.	138.
5.14	Applied load vs. strain gauge readings for the specimens of the three different storage conditions, Mix A.	144.
5.15	Applied load vs. strain gauge readings for the specimens of the three different storage conditions, Mix B.	144.
5.16	Applied load vs. loaded end slip for the modified pull-out test.	146.
5.17	Load distribution along the bar for the water saturated specimens.	147.
5.18	Load distribution along the bar for the sealed specimens.	147.
5.19	Load distribution along the bar for the oil saturated specimens.	147.
5.20	Bond stress distribution along the bar for the water saturated specimens.	148.
5.21	Bond stress distribution along the bar for the sealed specimens.	148.
5.22	Bond stress distribution along the bar for the oil saturated specimens.	148.
5.23	Bond stresses distribution along the bar for the water saturated specimens.	148.
5.24	Bond stress distribution along the bar for the sealed specimens.	148.
5.25	Bond stress distribution along the bar for the oil saturated specimens.	148.
5.26	Applied load vs. load at gauge position for water saturated, sealed, and oil saturated specimens.	151.
5.27	Applied load vs. load at gauge position for water saturated, sealed and oil saturated specimens.	151.
5.28	The formation of the rupture point along the reinforcing bar for the specimens of the three different storing conditions.	151.
6.1	Relative change in dynamic modulus (E_D) over that dried in oven for 14 days vs. age, Mix A_1.	158.
6.2	Relative change in dynamic modulus (E_D) over that dried in oven for 14 days vs. age, Mix A_2.	158.

Fig.No.	Title	Preceeding Page No.
6.3	Relative change in dynamic modulus (E_D) over that dried in oven for 14 days vs. age, Mix A_3.	158.
6.4	Relative change in dynamic modulus (E_D) over that dried in oven for 14 days vs. age, Mix B_1.	158.
6.5	Relative change in dynamic modulus (E_D) over that dried in oven for 14 days vs. age, Mix B_2.	158.
6.6	Relative change in dynamic modulus (E_D) over that dried in oven for 14 days vs. age, Mix B_3.	158.
6.7	Percentage change in weight of concrete prisms vs. age, Mix A_1.	162.
6.8	Percentage change in weight of concrete prisms vs. age, Mix A_2.	162.
6.9	Percentage change in weight of concrete prisms vs. age, Mix A_3.	162.
6.10	Percentage change in weight of concrete prisms vs. age, Mix B_1.	162.
6.11	Percentage change in weight of concrete prisms vs. age, Mix B_2.	162.
6.12	Percentage change in weight of concrete prisms vs. age, Mix B_3.	162.
6.13	Effect of storage condition and w/c ratio on the dynamic modulus of concrete, Mix A.	163.
6.14	Effect of storage condition and w/c ratio on the dynamic modulus of concrete, Mix B.	163.
6.15	Relative change in longitudinal resonant frequency (Fr) over that dried in oven for 14 days vs. age.	164.
6.16	Relative change in longitudinal resonant frequency (Fr) over that dried in oven for 14 days vs. age.	164.
6.17	Relative change in longitudinal resonant frequency (Fr) over that dried in oven for 14 days vs. age.	164.
6.18	Relationship between static and dynamic modulus of concrete stored at different conditions.	167.
6.19	Relationship between cube crushing strength and dynamic modulus for concrete stored at different conditions.	168.
6.20	Logarithmic decrement, δ vs. age, Mix A_1.	173.
6.21	Logarithmic decrement, δ vs. age, Mix A_2.	173.

Fig. No.	Title	Preceeding Page No.
6.22	Logarithmic decrement, δ vs. age, Mix A_3.	173.
6.23	Logarithmic decrement, δ vs. age, Mix B_1.	173.
6.24	Logarithmic decrement, δ vs. age, Mix B_2.	173.
6.25	Logarithmic decrement, δ vs. age, Mix B_3.	173.
6.26	The relationship between logarithmic decrement, (δ) and cube crushing strength for water and oil soaked specimens.	176.
6.27	The relationship between logarithmic decrement (δ) and dynamic modulus for water and oil soaked specimens.	177.

LIST OF TABLES

Table No.	Title	Page No.
1.1	Appendix 1 - Joint Committee report - effects of various oils on Portland cement concrete.	4.
3.1	Chemical composition of the ordinary Portland cement (OPC) used in this investigation.	43.
3.2	Fine aggregate grading.	44.
3.3	Coarse aggregate grading and properties.	45.
3.4	Analysis of crude oil.	46.
3.5	Details of concrete mixes used throughout this investigation.	49.
4.1	Details of the specimens to be tested throughout the investigation for each mix design.	54.
4.2	Compressive strength of HCPs and mortars at various ages, w/c ratios, and storage conditions - N/mm^2.	60.
4.3	Constants of regression - eqn. (4.2) compressive strength vs. w/c ratio for HCP and mortar.	62.
4.4	Elastic modulus (E) and failure stress (fc) for HCPs and mortars.	65.
4.5	Constants of regression - eqn. (4.3) elastic modulus vs. w/c ratio.	65.
4.6	Direct tensile strength (ft) and standard deviation (S.D.) for HCPs and mortars.	66.
4.7	Compressive strength of concrete cubes, cylinders, and prisms continuously cured in water for various ages - N/mm^2.	68.
4.8	Compressive strength of concrete cubes, cylinders and prisms of various ages, w/c ratio, and storage conditions - N/mm^2.	69.
4.9	Constants of regression - eqn. (4.6) compressive strength vs. w/c ratio.	72.
4.10	The splitting tensile strength of concrete at various ages, w/c ratios, and storage conditions - N/mm^2.	77.
4.11	The uniaxial tensile strength of concrete and the percentage of absorption at various ages, w/c ratio, and storage conditions - N/mm^2.	77.
4.12	Modulus of rupture of concrete at various ages, w/c ratios,and curing conditions - N/mm^2.	79.
4.13	Static modulus of elasticity and failure stress of concrete prisms continuously cured in water for different ages - N/mm^2.	82.

Table No.	Title	Page No.
4.14	The failure stress, critical stress (θ_{cr}) and the initiation stress (θ_i) for concrete specimens stored for 600 days at various storage conditions.	82.
4.15	Static modulus of elasticity of dried concrete specimens stored in different curing conditions - N/mm^2.	83.
4.16	Constants of regression - eqn. (4.8) modulus of elasticity vs. soaking period.	85.
4.17	Constants of regression - eqn. (4.9) elastic modulus vs. w/c ratio.	86.
4.18	The pulse-velocity (P.V.) and initiation stress (θ_i) for all mixes tested under uniaxial test (at 600 days).	89.
4.19	Properties of hardened cement paste used in this investigation.	93.
4.20	Typical calculation of mortar porosity.	94.
4.21	Typical calculation of concrete porosity.	94.
4.22	Constants of regression - eqn. (4.14) compressive strength vs. total porosity.	96.
4.23	Oil absorbed by HCP and mortar cubes after different soaking periods.	96.
4.24	Constants of regression (4.15) - absorption vs. w/c ratio.	99.
4.25	Constants of regression (4.16) - absorption vs. total porosity.	99.
4.26	Oil absorption (% by weight), variation of area saturated by oil, and the average depth of penetration with time and w/c ratio (using prisms of 100x100x500 mm.).	100.
4.27	Constants of regression - eqn. (4.17) absorption vs. time.	103.
4.28	Constants of regression - eqn. (4.18) depth of penetration vs. time.	103.
4.29	Constants of regression - eqn. (4.19) amount of absorption vs. porosity.	104.
4.30	Cube crushing strength as a % of the 1-day sealed strength.	110.
4.31	Reduction in compressive strength (per month) due to crude oil saturation.	111.
4.32	Comparison of the experimentally obtained elastic modulus with that predicted from CP.110.	127.
5.1	Details of concrete mixes used in the bond tests.	130.

Table No.	Title	Page No.
5.2	Bond strength of the oil soaked, sealed, and water saturated specimens for both plain and deformed bars.	136.
5.3	Average bond strength at different free end slip and the total free end slip at failure for both plain and deformed bars.	139.
5.4	The percentage of change in the bond strength as a percentage of sealed specimens tested at the age of 120 days.	142.
5.5	Percentage of oil absorption, the average ultimate bond strength, and the critical bond stress at free end slip of 0.025 mm for the modified pull-out test.	145.
5.6	Comparison of the experimental results and that given by the CP110 Code.	152.
6.1	The effect of composition, w/c ratio, and initial curing on the resonant frequency and the dynamic modulus of concrete prisms.	157.
6.2	Relative changes in the dynamic modulus and density with age to that of the oven dried samples - Mix A.	159.
6.3	Relative changes in the dynamic modulus and density with age to that of the oven dried samples - Mix B.	160.
6.4	Effect of w/c ratio on the dynamic modulus at different ages.	165.
6.5	Constants of regression - eqn. (6.1) dynamic modulus vs. w/c ratio.	165.
6.6	Constants of regression - equation (6.2) static modulus vs. dynamic modulus.	169.
6.7	Comparison of the experimentally obtained dynamic modulus and that predicted from CP110.	170.
6.8	The logarithmic decrement ($\delta \mathrm{x} 10^{-2}$) at the age of 7 days and after being oven dried.	170.
6.9	Change of logarithmic decrement ($\delta \mathrm{x} 10^{-2}$) with age for concrete specimens soaked in crude oil, water, and sealed as a reference.	172.

LIST OF PLATES

Plate No.	Title	Preceeding Page No.
3.1	Hobart bench mixer.	47.
3.2	Testing a briquet for direct tensile strength of HCP and Mortar.	48.
3.3	Direct tensile test for concrete specimen.	50.
3.4	Crude oil pressure vessel.	52.
4.1	Penetration of oil through the concrete cubes as shown under U.V. lamp.	101.
4.2	Penetration of oil through the concrete cylinder as shown under U.V. Lamp.	101.
4.3	Penetration of oil through the concrete cylinder (splitting test) as shown under U.V. Lamp.	101.
4.4	Compression test-type of failure for cube cylinder and prism concrete specimens.	113.
5.1	Detail of the pull-out test specimens.	130.
5.2	Instrumented bar preparation and gauge location for modified pull-out test.	132.
5.3	Modified pull-out test using Avery universal testing machine.	133.
5.4	Crack patterns of pull-out test using deformed bar.	134.
5.5	Penetration of oil through the pull-out specimens as shown under U.V. Lamp.	141.
5.6	Modified pull-out test, type of failure.	144.
6.1	Test set-up for fundamental resonance in longitudinal vibration.	153.

ABBREVIATIONS

OPC	Ordinary Portland Cement.
HCP	Hardened Cement Paste.
w/c	Water/CementRatio.
OVD	Oven Dried to 105° C for 14 days.
SE	Sealed in a polythene bag.
SO	Soaked in Oil.
SW	Soaked in water.
C-S-H	Calcium Silicate Hydrate.
SSD	Samples stored in Water and Tested in Saturated Surface Dry Condition.

CHAPTER 1.

INTRODUCTION

1.1 GENERAL INTRODUCTION.

Structures for the storage or transportation of oil have for years been constructed in steel(1,2,3,4), but as a result of the critical shortage of steel plate and problems of serviceability and safety during the second world war, concrete tanks, reinforced or prestressed, were used to store many different liquids. Owing to the seepage of oil through the concrete and leakage at the joint, successful attempts(5,6) to store oil in concrete tanks were only made recently and many concrete tanks for oil storage are in use underwater, e.g. at the Ekofisk-field(7) and the Shell-Esso Brent offshore oil field(8).

Prestressed concrete tanks(1,2,3) constructed in the past have performed satisfactorily under many varied conditions and their advantages include shock and fire resistance, cheap maintenance, and the fact that they may be built to much larger dimensions than steel tanks. Concrete tanks, however, have some disadvantages, i.e. the unknown behaviour of concrete in direct contact with hydrocarbons, leakage or contamination of the liquid, construction difficulties due to the need to prevent differential settlement and finally the difficulties of any modifications and repairs.

However, the action of oil on concrete is still obscure and the recent vast growth of crude oil production from the offshore fields (North Sea) and the Middle East countries, together with the problems arising from transportation and storage necessitate a quantitative study of the properties of concrete in contact with oil.

1.2 THE EFFECT OF OIL ON CONCRETE.

Mineral oils(9) and fuel oils do not attack matured concrete because they contain no constituents which react chemically with hardened cements. Lubricating oils which are entirely of mineral origin do not harm concrete structures, but if vegetable oils or animal oils are present, they have a definite action and damage has been recorded in some structures.

The aggressivity of the oils(2,3,10) depends on their viscosity, so that the higher the viscosity of the oil, the less dangerous it is to concrete. The viscosity of oil is thus an important factor. Storage of mineral oils in concrete tanks(9,10) has shown that concrete can be made sufficiently impermeable to stop seepage in the case of fuel oils or heavier fractions. The lighter products with low viscosity(9,11), on the other hand, will easily penetrate into or through the concrete. Oils such as diesel oils, light fuel oils, kerosene and gasoline, which have viscosities lower than 6 centipoises at 21° C, might leak(3,4,11,12) excessively from unlined concrete tanks.

Aviation gasoline(1,13) in particular presents the greatest problem since it contains toluene, zylene and benzene which react with free alkali in the concrete and cause deterioration of the concrete itself and of the stored gasoline. Thus it appears justifiable to conclude that unlined concrete tanks should be used only for storage of gasoline intended for immediate use, and should not be used for prolonged storage, nor should gasoline from concrete tanks be stored again, even in steel tanks for a prolonged period before use.

Pure benzene (petrol) is not aggressive(10) to concrete but is capable of seeping through the densest concrete. In addition to having a high seepage velocity, benzene sometimes contains acidic polluting agents. Therefore, concrete tanks, even if a sealing mortar rendering is applied,cannot be made for the storage of benzene.

There are results,however, which show that concrete deteriorates when in contact with crude oil especially if saturated by the oil. Biczock(10) recommended a highly compacted concrete with a well graded aggregate and sufficiently rich mix for machine foundation. He found that a concrete sample from an oil-saturated floor containing no more than 40 litre of oil per cu-m of concrete, shows no reduction in strength. In comparison, samples containing 100 litre oil per cu-m concrete show 40% reduction in strength. Concrete will regain its strength in proportion to the amount of oil removed if it is allowed to dry.

Meissner(14)reported some small reduction in the compressive strength of 2-in mortar cubes soaked for 180 days in high octane gasoline. Hansen(15) found that the creep rate of concrete saturated with paraffin oil after 5 years increased much faster than that of concrete in air or water and also observed lower strength for oil saturated specimens. On the other hand, Appendix 1 of the Joint Committee Report(16), which gives the effects of various oils and similar liquids on Portland Cement concrete,suggests no serious effect of the penetrating oil on the properties of unprotected concrete. That part of the appendix relating to oils is quoted in table (1.1).

Substance	Effect on Unprotected Concrete
Light fuel oils above 30° Baume	None - some loss of oil from penetration
Volatile oils - kerosene, benzene, naphtha, gasoline	None - considerable loss of oil from penetration.
Heavy oils - 30° Baume or heavier	None - very slight penetration.

Table (1.1) Appendix 1 - Joint Committee Report Effects of various oils on Portland Cement Concrete.

Bahador, A.R.(17) studied the physical properties of concrete and mortar containing various quantities of oil admixture. He found that the crushing strength of oil-mixed concrete is lower than that of plain concrete. Concrete with 10% of oil has 75% of the strength of plain concrete at 28 days. The tensile strength of oil-mixed cement mortar (1:3) is very little different from that of plain mortar, and shows a substantial gain in strength at 28 days and 6 months over that at 7 days. Bond between plain bars and concrete is seriously affected by the mixture of oil. The bond of deformed bars is not so seriously effected, but is somewhat decreased by the oil admixture. The time of initial and final set is delayed by the addition of oil.

The author, in his dissertation submitted to the University of Sheffield for the degree of Master of Engineering(18), studied some properties of oil soaked concrete. It was found that the compressive strength and the tensile strength of oil soaked concrete specimens were reduced by at least 9% and 9.5% respectively. This reduction as shown in fig. (1.1) and fig. (1.2), is related to the degree of saturation which in turn is related to

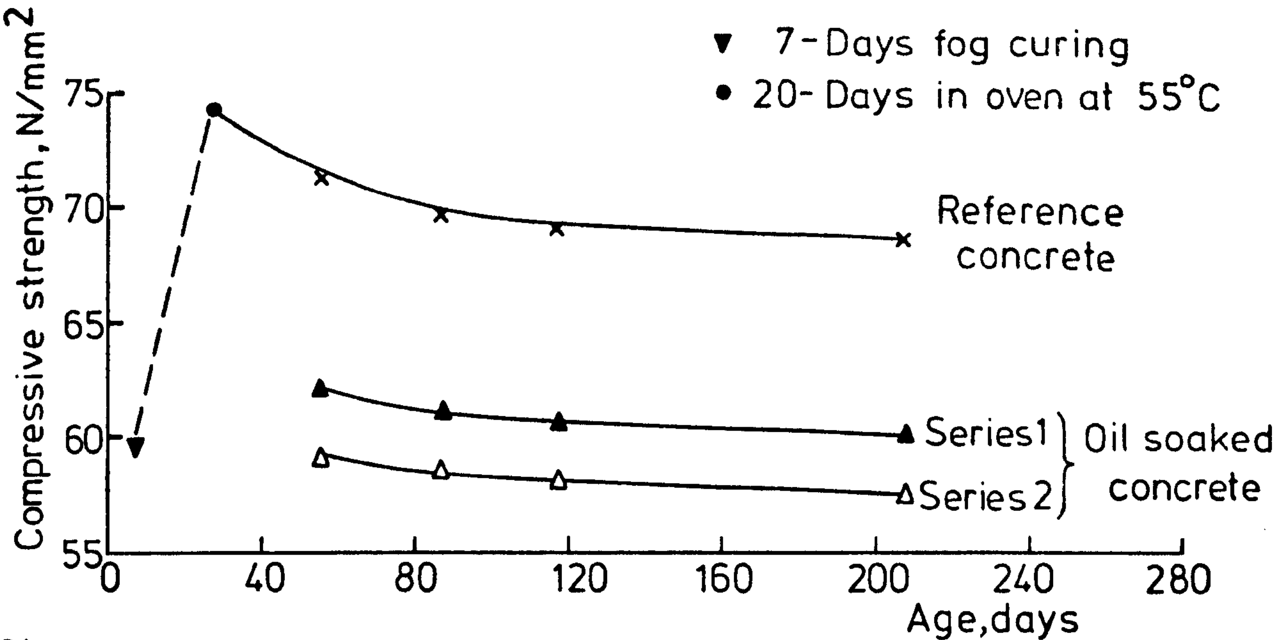


FIG. 1-1 THE EFFECT OF ABSORBED OIL ON THE COMPRESSIVE STRENGTH OF CONCRETE

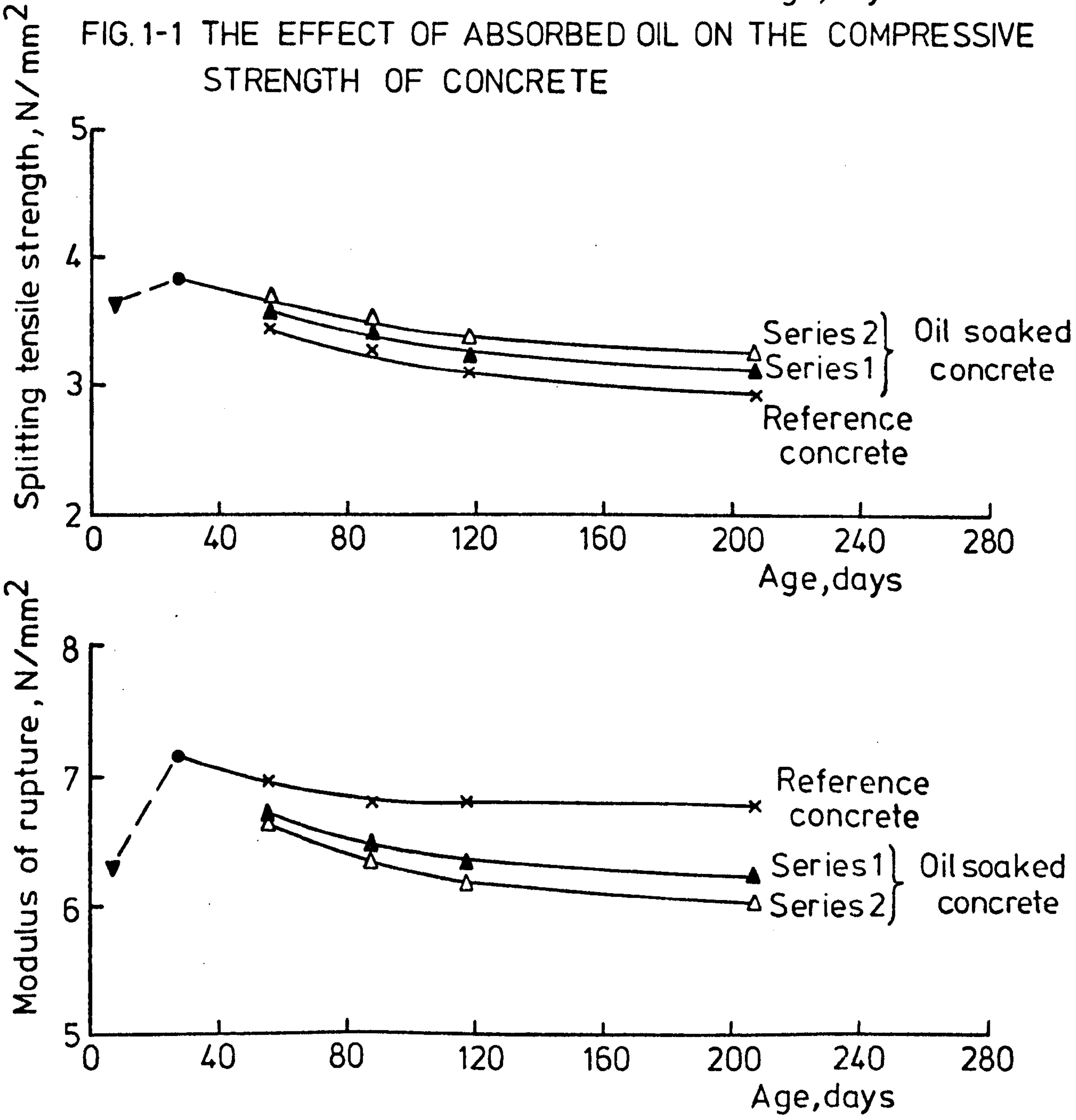


FIG. 1-2 THE EFFECT OF ABSORBED OIL ON THE TENSILE STRENGTH OF CONCRETE

the length of the soaking period and the porosity of concrete. Another noticeable result is that the splitting tensile strength of oil soaked specimens was higher than the companion sealed specimens tested at the same age. This increase as shown in fig. (1.2) was also related to the degree of oil saturation and the length of soaking period. The explanation of this variation in tensile strength between oil soaked and sealed specimens, was thought to be due to the closing and autogenous healing of the cracks formed during hardening when the specimen was soaked in oil. Alternatively, it might be due to some reduction in the tensile strength of the unsoaked specimens due to the presence of such cracks.

On the other hand, the bond strength of the oil soaked specimens was reduced by 3-8.5% for mild round steel bars and 3.5-5% for deformed bars. This reduction was related to the amount of absorbed oil and length of soaking period. The variation in bond strength of the oil soaked specimens was thought to be due to the reduction in the frictional resistance in the case of plain bar, and due to the weakening effect on the strength of concrete between the ribs as well as the reduction of the frictional resistance in the case of deformed bars.

1.3 PURPOSE OF THE PRESENT RESEARCH.

1. A study of the effect of absorbed crude oil on the mechanical properties of hardened cement paste, mortar, and concrete. This involves the measurement of compressive strength, tensile strength, static modulus of elasticity and volumetric strains under compressive loads, for the dry specimens soaked in oil, soaked in water, and sealed as a reference.

2. A study of the shrinkage strain of oil soaked concerete compared to that of sealed specimens or those soaked in water for different soaking periods up to 550 days.

3. A study of the dynamic modulus and the damping capacity of oil saturated specimens soaked for different periods.

4. A study of the effect of different degrees of oil saturation on the average bond strength for both plain and deformed steel reinforcing bars.

5. A study of the bond stress distribution along a plain bar in a dry concrete specimen soaked in oil for eight months.

6. A study of the effect of different concrete porosities on the amount and depth of penetration of absorbed oil.

1.4 SCOPE OF THE INVESTIGATION

Matti(19,20) and Oyeka(21) have carried out intensive investigations into the effects of crude oil on the permeability, shrinkage and porosity of concrete. The present investigation is a continuation of on-going studies in this department on the behaviour of oil soaked or saturated concrete with a view to making a more efficient use of concrete for oil storage tanks.

An extensive study of the mechanical properties of hardened cement paste, mortar and concrete for oil soaked specimens is made in Chapter 4. Different porosities and soaking periods were investigated. Detection of cracks for oil saturated specimens under compressive load was investigated by using the pulse velocity technique.

A study into the variation of bond strength with the amount of absorbed oil is made in Chapter 5. Concentric cylindrical pull-out specimens reinforced with only one central bar, deformed or plain, were tested after different oil soaking periods. A modified pull-out test was designed to measure the bond stress distribution along a plain bar in concrete prisms saturated with crude oil.

In Chapter 6, the deterioration due to soaking in oil was investigated by measuring the dynamic and the damping capacity of concrete prisms. The relationships between these two properties and other mechanical properties are also presented.

The conclusions are presented in Chapter 7, together with recommendations for further work.

CHAPTER 2.

LITERATURE SURVEY.

2.1 THE STRUCTURE OF CEMENT PASTE AND CONCRETE.

Fresh cement paste is a network of particles of cement in water, it is plastic and normally remains thus for an hour or more, after which this plastic mass sets. In the first few minutes of mixing, the reaction rate is rapid and calcium silicate hydrate-cement gel (C-S-H) forms a coating around the cement grains. As hydration progresses, hydration products gradually fill the capillary pores formed. Early hydration is confined to the narrow pores producing tobermorite crystallite which is a stronger material and acts as a solid link between the coated cement particles, so producing a continuous solid matrix within the cement paste.

Hardened cement paste can thus be described(22,23) as a multi-phase material composed of unhydrated cement particles embedded into a continuous matrix of cement gel which, in turn, is a matrix of semi-amorphous intertwined fibrous or needle-shaped particles, and thin crumpled sheets and foils which form a continuous matrix having a continuous system of minute water-filled voids called gel pores. Cement gel is inter-penetrated by capillary pores or cavities filled with water to a degree dependent upon the amount of drying.

Concrete(22), however, is a very complex multi-phase material composed of at least seven components, i.e. coarse aggregate, sand, unhydrated cement particles, cement gel, capillary and gel pores, and pore water and accidentally or deliberately entrapped air voids. Concrete(23) can also be considered as two-phase material consisting of a uniform distribution of coarse particles in a homogeneous matrix of different composition.

2.2 THE STRENGTH OF CONCRETE.

A primary function of practically all structures is to carry load or resist applied forces of whatever nature; other functions such as the retention of fluids, the exclusion of weathering or other destructive agencies may be involved also. The strength of concrete appears to be a good index, direct or inverse, of most of the other properties of practical significance.

Failure of concrete(24) is, in general, a progressive process, and so failure of the paste close to a single particle of aggregate need not cause collapse of the concrete but will increase the stress in much of the remaining unbroken parts of the specimen. As the applied load is increased, the number of aggregate particles around which paste failure has occurred increases until a stage is reached when the unbroken paste and aggregate framework can no longer support the applied load and collapse occurs.

Newman(25) suggests that the prediction of the overall properties of a composite concrete system requires a knowledge of the volume fractions and properties of the constituents, as well as the mechanism of interaction between the separate phases. Hobbs(26) states that the strength of concrete may be governed by one or more of the following strengths;

1. The strength of the cement paste,
2. The strength of the aggregates and,
3. The bond strength between cement paste and aggregate.

2.2.1 The Cement Paste Strength.

Grudemo(27) found that when cement grains start hydrating, a coarsely fibrous structure of slender rods grows radially from the grains in the outer solution to fill all interstitial space. In the areas of contact between cement particles, the rodlike formations grow in between each other and bonds are formed which are steadily increased. The bonds in these contact points are probably one of the sources of strength and rigidity of cement paste.

Powers and others(28,29,30) believe that the actual source of strength of the gel probably arises from two kinds of cohesive bonds. The first type is the physical attraction between solid surfaces, separated only by the small gel pores; this attraction is usually referred to as the van der Waals force. The source of the second type of cohesion is the chemical bonds of the gel particles, which are cross-linked by chemical forces. The physical attraction of van der Waals forces hold the particles in positions of least potential energy and are considered to be the most important. The chemical forces, on the other hand, are much stronger than the physical forces but cover only a small fraction of the boundary of the gel particles.

Philleo(31) considers the strength as being a result of secondary bonding (van der Waals) and explains the increase in strength with hydration as being caused by the increased surface area over which the bonds act. Feldman and Sereda(32) consider the bonding force as interparticle bond and consider chemical bond to be very insignificant. The nature of the interparticle bond is not certain but is not van der Waals forces since it is independent of surface area but is influenced by interlayer water.

Powers and Brownyard(28) have shown experimentally that strength increases as the cube of the gel-space ratio, where gel-space ratio is the ratio of gel volume to gel volume plus capillary space. The gel concentration is a function of the original water-cement ratio and the degree of hydration. Since a unit volume of cement produces 2.2 units of gel, complete hydration cannot be approached if the w/c ratio is less than 0.38 by weight. At higher water-cement ratios, the volume of the hydrate produced is insufficient to fill the space available and the remaining void space is called 'capillary pores'. These pores may or may not form a continuous system and have the effect of diluting and thereby weakening the gel.

2.2.2 The Aggregate-Cement Bond Strength.

Bond strength(33) between cement paste and aggregate may be thought of as the force required to separate two solid components in concrete at their interface or alternatively, it can be thought of as the effective adhesive force holding two surfaces together. This force is derived(34,35) from two sources, one being chemical and the other mechanical. As a first approximation, aggregates are often considered to be inert, in which case the bonding forces across the aggregate-paste interface will be mostly physical or adhesive in nature, of the van der Waals type. Other evidence(34) suggests that aggregates are not completely inert and chemical reactions between aggregate and cement paste may produce chemical or cohesive bonds of higher strength.

Bond between coarse aggregate and the cement paste or mortar matrix is usually considered to be the weakest link in the heterogeneous concrete system(34,35,36). The adhesive strength of the bond between aggregate and cement is often lower than the cohesive strength of the cement matrix. Even with the same cement and aggregate, and same water-cement ratio, bond strength can range from a fraction of paste strength to a value greater than the paste strength. The ratio of bond to paste strength varies widely with the temperature and duration of curing(34).

However, the strength of the bond between aggregate and matrix(36) depends not only upon the pure adhesion but also upon the degree of mechanical interlocking, i.e. it depends on the one hand upon the roughness and shape of the particles and their packing, as well as the strength of the matrix. Jones and Kaplan(37) concluded that in compression,precracking probably arises from local breakdown in the adhesion between the coarser aggregate and the cement and that concrete containing smooth gravel aggregate begins to crack at lower stresses than concrete containing aggregate with a coarser surface texture.

2.3 INTERNAL MICROCRACKING IN CONCRETE.

Microcracking is inherent in concrete. When still fresh, cracking can occur due to rapid evaporation of water during placing which leads to excessive shrinkage,and the separation of aggregate and matrix due to settlement of the later and to local bleeding under pebbles. After hardening, concrete develops cracks in one of the following ways:-

1. locally restrained shrinkage of the matrix may cause bond failure between it and the aggregate.
2. shrinkage may be restrained by end conditions or element size, resulting in large scale tensile cracks.
3. severe temperature fluctuations may cause excessive shrinkage on drying and reduction in surface tension on wetting, both of which are conducive to cracking.

These microcracks have an influence on all concrete mechanical properties, including deformation and strength, whether instantaneous or time-dependent.

The basic mechanism (22,33) of failure of mortars and concrete under load is the initiation and propagation of small or microscopic cracks (microcracks) which extend and interconnect until, at ultimate load, the whole internal structure is completely disrupted. The load stage at which severe, detectable cracking begins has been termed a discontinuity by Newman (38).

The presence of microscopic cracks and the progression of internal splitting in concrete specimens in compression was first suspected by Brandtzaeg (39) in 1929. He observed the volumetric changes of plain concrete cylinders in concentric compression and found at a critical load, 77-85% of the maximum load, the volume started increasing rather than continuing to decrease. His conclusion was that failure progressed by internal splitting in microscopic regions distributed throughout the material. In the same work he summarized that the shape of the stress-strain curve was related to this internal splitting, and that the curvature was due to rapid expansion of the material.

Thereafter, several techniques have been used to observe or infer the presence of microcracks. Small hand microscopes and extensometers were used as early as 1936, but these techniques were tedious and limited to the surface cracks. Slicing the specimen after applying a given load was carried out to observe internal microcracks. This method revealed the location and length of the cracks but, because of the loss of continuity of the specimen inherent in a slicing technique, the propagation of microcracks cannot be observed from one load to the next. In 1952, Jones(40) inferred the presence and development of microcracks from changes in velocity of an ultrasonic pulse. Recently, X-ray techniques for crack detection have been developed in the United States by Slate and Olsofski(41).

Most of these techniques have been used in short-term static loading of concrete with the primary object of explaining the deformation behaviour as reflected in the stress-strain curve. As a result of the observations(33,38,42,43,44) that have been made, the following statements would be generally agreed:-

1. Bond cracks exist at the interface between coarse aggregate and mortar before concrete is subjected to any external load, even in concrete kept continuously wet from the time it is cast.
2. The curvature of the stress-strain curve above 30-50% of the ultimate load is mainly due to microcracks which begin to increase appreciably in length, width and number with increasing strain.
3. At about 70-90% of the ultimate load, the number of cracks through the mortar begins to increase noticeably and, by bridging between bond cracks, continuous crack patterns beging to form. The development of a continuous crack pattern does not itself

lead to the immediate loss of carrying capacity, because microcracked concrete constitutes a highly redundant stable structure until it reaches the peak of the stress-strain curve. However, when a continuous crack pattern has developed extensively, the carrying capacity of concrete decreases and the stress-strain curve begins to descend.

2.4 FAILURE OF CONCRETE TEST SPECIMENS.

2.4.1 Strength in Compression.

The mode of failure of concrete in compression is illustrated in fig. (2.1) which shows a typical stress-strain behaviour(45). Up to a certain proportion of the ultimate stress, the strain is linearly proportional to the stress. When the critical stress - θ_c - is exceeded, both the longitudinal and lateral strain increase at a faster rate than the stress and the volumetric strain decrease. Ultrasonic(40) and X-ray measurements(41) indicate that at this point of deviation from linear elastic behaviour considerable microcracking of the specimen occurs in a direction parallel to that of the compressive load. Rapid failure, under tensile states of stress, occurs shortly after the bond cracks begin to extend into the mortar matrix.

The normal method of loading a cube specimen through steel platens generally(46,47,48) produces a non-uniform stress distribution through the height of the specimen and induces frictional forces at the interface of the specimen and the platen, which prevent the ends of the specimen from expanding laterally. Because of friction, an element within the specimen is subjected to a shearing stress as well as to compression, the magnitude of the shearing stress decreases and the lateral expansion increases with an increase in distance from the platen. As a result of the restraint, in a specimen tested to destruction there were zones of sound concrete at each end of the specimens, these were pyramidal in cubes and prisms and conical in cylinders.

When the friction is eliminated, by the placing of different material (46,47,48) between the ends of the specimen and the platens of the machine, the specimen exhibits a large lateral expansion of the end faces. The failure in such a specimen is governed by the lateral strain so that when this strain exceeds the ultimate tensile strain of concrete, failure will occur by splitting at right angles to the direction of the load, and this has been frequently observed, especially in specimens whose height is greater than their breadth. This is dependent mainly upon the tensile bond strength at the aggregate-paste interface. The tensile cracks first propagate around the aggregate-paste interface and then interconnect through the mortar matrix. However, there is obviously a transition from tensile bond failure to shear bond failure at the aggregate-paste interface as the applied state of stress becomes more compressive.

Jones (43) suggested that the initial flaws in the specimen act as nuclei from which the microcracks propagate. The component of the compressive stress in the direction of loading tends to close down those initial flaws which are nearly at right-angles to the direction of loading. However, localized tensile or shear forces arising from the differences between the properties of the aggregate and of the cement paste tend to cause rupture and spread of microcracks along those aggregate-mortar boundaries which are parallel or nearly parallel to the direction of the load.

Glucklich$^{(49)}$ has discussed the failure of compression specimens from the standpoint of Griffith's theory of crack propagation. He defines two types of cracking, namely first-crack type and progressive-cracking type. The former arises in the case of specimens with lubricated end corresponding to specimens tested with rubber insertions as discussed above and is considered a premature fracture. Progressive cracking occurs under normal and restraint conditions and Glucklich has discussed possible mechanisms governing the propagation of the cracks in this case. Final failure under normal restraint conditions is considered to consist of progressive cracking followed by diagonal shearing of the weakened zones of the concrete.

Therefore, several investigators have suggested that under uniaxial compression, the formation of cracks at the phenomenological level is usually the result of a cleavage splitting failure in the direction of the maximum compressive stress. On the other hand, the work on shear bond strength suggests that in compression these cracks are initiated at the structural level, by a shear bond failure, and the cracks are first formed by a local shear bond failure but then propagate parallel to the direction of compression. The probable reason for this is that the plane of compression is the only plane which does not have a normal component of compressive stress acting on it, and is therefore the plane with the least cohesive bond strength.

2.4.2 Strength in Tension.

Although concrete is not normally designed to resist direct tensile forces because of its relatively low tensile strength, the knowledge of tensile strength is of value in estimating the load under which cracking will develop. These values can be obtained from three different types of test as described in the following sections.

2.4.2.1 The Splitting Tensile Test.

The indirect tensile test(50,51,52) which involves splitting a cylindrical specimen under diametral compression has provided a satisfactory method of determining the tensile strength of concrete. It is simple to perform and gives more uniform results than other tension tests, but it is relatively more variable than that of the compression test. The strength determined in the splitting test is believed to be closer to the true tensile strength of concrete than the modulus of rupture. Another advantage is that the same type of specimen can be used for both the compression and the tension tests.

Other types of splitting tests(53,54) introduced subsequently are the cube split test, the diagonal cube split test and the prism test. In all these tests, compressive line loads are applied along a vertical symmetrical plane setting up tensile stresses normal to the plane which causes the splitting of the specimen.

The nature of failure in the cylinder splitting test is characterised by a sudden tear along the loaded vertical plane accompanied sometimes, by a loud noise. A concrete cylinder is laid horizontally between the loading platens of the testing machine and the load is applied along the generatix. Then an element on the vertical diameter of the cylinder, as shown in fig. (2.2) is subjected to both compressive and tensile stresses acting in mutually perpendicular directions which indicates a

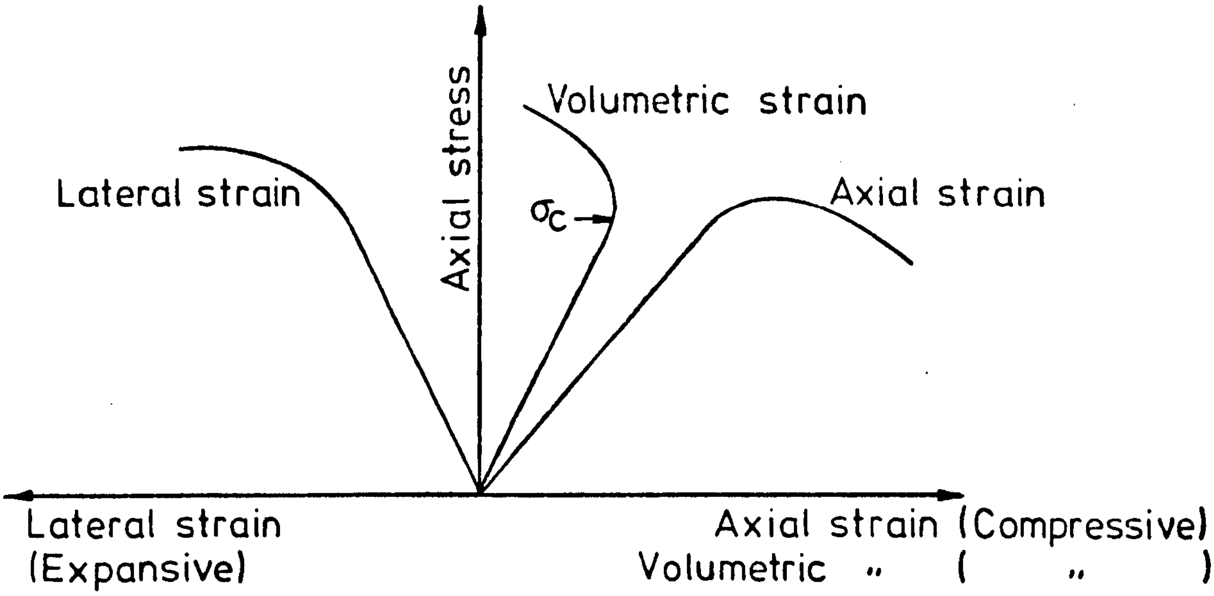
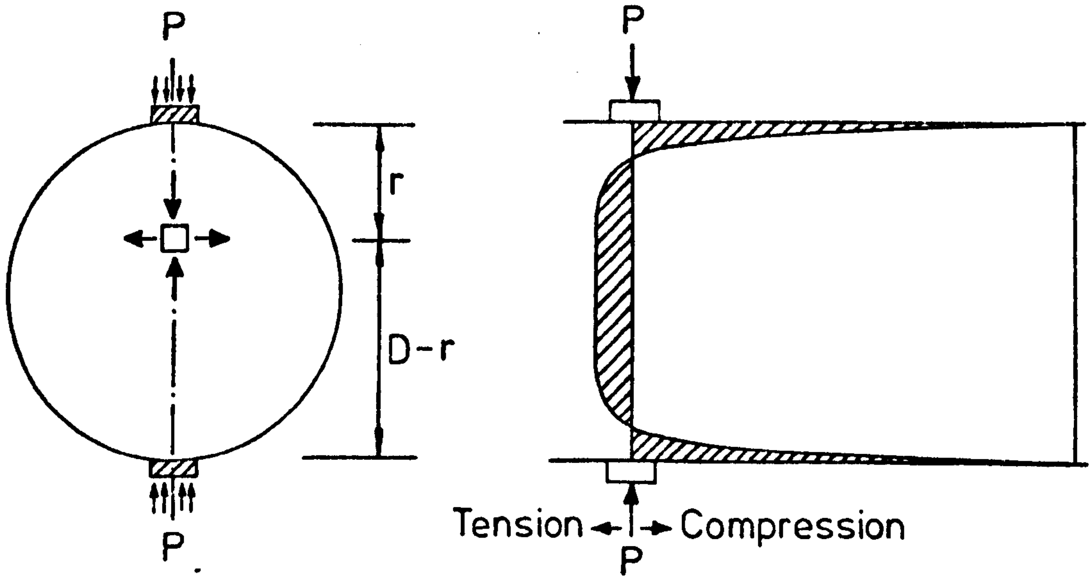


FIG. 2·1 TYPICAL STRESS-STRAIN BEHAVIOUR OF CONCRETE OR MORTAR UNDER COMPRESSION



(a) Cylinder split test

(b) Distribution of horizontal stress in a cylinder loaded over a width equal to D/12

FIG. 2·2 STRESS DISTRIBUTION NORMAL TO THE LOADED PLANE FOR SPLIT TEST SPECIMEN biaxial state of stress on the failure plane(52,55). Their values are given by the following equations:-

$$\text{vertical compressive stress} = \frac{2P}{\pi LD}\ \frac{D^2}{r(D-r)} - 1 \quad \text{.....(2.1)}$$

$$\text{horizontal tensile stress} = \frac{2P}{\pi LD} \quad \text{.....} \quad \text{(2.2)}$$

where;

P = The compressive load on the cylinder.

L, D = Length and diameter of the cylinder respectively.

$r, D-r$ = The distances of the element from the two loads respectively.

2.4.2.2 The Flexural Test.

In such tests, prisms are subjected to central or third-point loading. The theoretical maximum tensile stress reached in the bottom fibre of the test prism is calculated and known as the modulus of rupture. This calculation might be unrealistic(54,56) due to the assumption of linear stress-strain relationship, while the shape of the actual stress block under loads nearing failure may not be triangular.

The method of failure of concrete in flexural tests is thought to be the result of progressive development of micro-cracks at the boundaries of particles of coarse aggregate, leading first to a fairly uniformly cracked tension face and finally to the slow propagation of the fracture crack. The final fracture crack generally links existing cracks and its path may traverse the different zones of cement paste, mortar and the boundaries of coarse aggregate or the aggregate particles themselves. The development of this path depends(57) on certain factors such as the strength of the paste,the size, surface texture and strength of both the fine and the coarse aggregate particles, the moisture condition of the specimen and also the rate of loading.

It is also assumed that (56) a beam can be divided into a number of elements, in such a way that the beam will fail when any one element is subjected to stress equal to its own strength. In the case of a beam under third-point loading, the central third is made of (N) elements. The elements at the same depth are subjected to the same stress and the beam will just fail at a loading giving rise to a stress equal to the strength of the weakest element. Therefore, in the case of third point loading a beam may be expected to fail at the weakest point in the middle third.In comparison,when central loading techniques are used,the opportunity for the weakest point to effect the result is much decreased.

2.4.2.3 The Uniaxial Tensile Test.

The indirect tensile tests have been generally finding preference on account of the many technical difficulties involved in conducting direct tensile tests. These tests give a gross over-estimate of the tensile strength of concrete because of the complex state of stresses in the specimen, and hence are of little use in studying the failure mechanism and microcracking of concrete under tensile loading. It may be reasonably assumed that the state of stress in the direct tensile test is less complex than that in the indirect tensile test and the tensile strength obtained is closer to the true value of tensile strength as defined by the average stress of an axially loaded long concrete specimen.

The basic difficulty in making a direct tensile test is to achieve a uniform distribution of tensile stress at a section of the concrete without inducing high stress concentration elsewhere. Clamping the specimen in the testing machine and applying a uniform tensile load to the specimen is difficult, and requires careful attention. Direct tensile tests(58,59) conducted on briquettes, bobbins, cylinders, or prisms have all suffered from local stress concentration. The use of an epoxy adhesive(60) instead of a clamping device offers some advantages. However, it has been shown(60) that even with the use of an adhesive, appreciable stress concentrations can still occur at the ends of the specimen adjacent to the adhesive. A necked test specimen was suggested to avoid the possibility of failure near the adhesive-concrete interface.

The mode of failure of a specimen in direct tension is considered to be much simpler than that of a specimen in compression. It is generally considered that failure in tension is initiated at the paste-aggregate interface because dense aggregates are relatively strong in tension and because the paste-aggregate bond strength is lower than the paste strength(34). As the tensile load increases, initial flaws in the concrete extend along the boundaries between mortar and coarse aggregate and subsequently a propagating crack is formed across the intervening mortar or aggregate phase. Unlike flexural-strength tests under one central load and splitting strength tests in which the strength is determined from the load-carrying capacity of a preselected section, specimens subjected to direct tensile-strength test fail in the cross section with the least load-carrying capacity.

The direct tensile test(59), however, is unsuitable for routine testing and tends to give a wide variation of results, and the ultimate load may be affected by testing machine stiffness and stress concentrations in necked specimens.

2.5 EFFECT OF DRYING ON THE STRENGTH OF CONCRETE SPECIMENS.

The moisture content(61,62,63) is important to the physical behaviour of concrete. The water can be placed in two categories conveniently described as:-

1. Evaporable water: free water which is held in the capillary channels and pores of the concrete and beyond the influence of the surface forces of the gel. It can be removed completely by drying at 105° C without disturbing the cement gel structure.
2. Non-evaporable water: consists of the water fixed chemically during the hydration of cement, and of water fixed by hydration products, which does not evaporate when oven dried at 105° C, but escapes gradually when heated from 105° C to 550° C in a furnace.

A rise in temperature causes(62) an accelerated drying and emptying of the gel and capillary pores, a decrease in the adhesion forces between the solid particles and the confined liquid layers, and an increase in the rate of hydration. Therefore, the property of concrete depends on the moisture content and its variation, so a knowledge of its moisture content in the drying process is important.

Pihlajavaara(64) shows that the dry strengths were about two thirds higher than wet strengths. He suggests that the change in moisture content alters the strength of porous solids. Generally speaking, drying causes strengthening and wetting creates a weakening of porous media. On the basis of published results, he assumed that the ratio of wet strength to dry strength of various concretes seems to vary from 1:1.2 to 1:2. Thoroughly dried specimens(65,66) on the other hand, suffered a sharp reduction in strength upon being immersed in water, and the strength returned to approximately that of the continuously wet cured control specimens. This reduction was thought to be due to the dilation of cement gel by absorbed water causing a reduction in the forces of cohesion between the solid particles and a consequent decrease in cement paste strength.

Temperature variation(62.66) influences tensile strength, modulus of elasticity, and bond strength development of concrete in much the same manner as it does compressive strength development. Compressive strength, tensile strength, modulus of elasticity and bond strength are all related, and an increase in one is reflected similarly in the others. But Johnston and Sidwell's work(65) suggests that the different effects produced by drying depend on the drying conditions and the type of test. For example(67), the splitting tensile strength of a concrete cylinder subjected to rapid drying in an oven at 140° C for various times ranging from zero to 14 hours,was significantly lower than the tensile strength of saturated concrete. The tensile strength of completely dry concrete was significantly greater than the tensile strength at intermediate saturation levels during drying.

Flexural strength tests are highly sensitive to change in moisture. Bloem and Walker$^{(68)}$ and Waters$^{(69)}$ observed that, the breaking load was sharply reduced by partial drying of saturated beam specimens. It could be reasoned that drying of the specimen was detrimental during the early periods, with initial shrinkage of the surface inducing tensile stresses in the outer fibres and reducing the load carrying capacity. Resoaking would tend to relieve the surface tensile stresses at early ages and increase the measured strength. It was also observed that complete (as opposed to partial) drying produced an increase in the measured flexural strength from which it could be reasoned that the differential shrinkage disappeared and the dry concrete had a higher strength. Resoaking the specimens dried in this manner reduced the strength after drying had become uniform, which might be due to the dilation of cement gel by absorbed water,as explained earlier.

2.6 VOLUME CHANGES OF CONCRETE ACCORDING TO MOISTURE MOVEMENT.

It is well known that concrete shrinks when it dries and swells when it is saturated with water and hence drying shrinkage of concrete is defined as the deformation due to the withdrawal of moisture from concrete in the absence of applied load$^{(70,72)}$. All concrete elements and structures are subjected to volume changes in varying degrees dependent upon the shape and environment surrounding the concrete. Local consequences of shrinkage are the generation of residual stresses, which may exceed the tensile strength of concrete, thus causing microcracks that make concrete unsuitable for storage purposes.

Extensive research(70-75) work has for years been carried out on this subject by many researchers and there is a fairly clear understanding of the drying shrinkage and swelling of concrete.

2.6.1 Factors Effecting the Drying Shrinkage of Concrete.

When concrete dries, the moisture content decreases at a gradually reducing rate as the concrete seeks to attain hygral equilibrium with its environment. If the relative humidity of the environment is sufficiently high, drying shrinkage is inhibited since there is no evaporation of water. Campbell-Allen(76) has suggested that drying shrinkage will occur at relative humidities below 94%. According to Davis and Kelly(77) any drying shrinkage that occurs at relative humidities of 95% or more is so small that it is offset by expansion due to the continued hydration of cement. The observation on shrinkage at various relative humidities has shown that the lower the relative humidity, the higher the drying shrinkage of concrete(78,79). At the same time, for a small concrete member the drying may be largely completed in a year or two, but for a large member it will take a long time. Drying shrinkage is therefore influenced by the size and shape of the member as conclusively demonstrated by Ross(80) and Hansen and Mattock(81).

The prime active component controlling the shrinkage of concrete is the cement paste which consists of cement and water. The amount(82,83) of water per unit volume of concrete is the most important single factor affecting shrinkage. Alexander(84) and others(85,86) have shown that the water/cement ratio is the main parameter affecting the shrinkage. Picket(87) found that the initial shrinkage is proportional to the w/c ratio, while shrinkage occurring on further storage in water and drying is independent of the w/c ratio. The quantity of cement per unit volume of concrete, on the other hand, appears to have an erratic effect on shrinkage. Washa(88) stated that, for a given material and a constant water content, the cement content has only a minor effect on drying shrinkage. The properties of cement such as composition and fineness etc. appear to have little influence on the ultimate shrinkage of concrete, though the fineness has some influence on the initial rate of shrinkage(76).

The grading, composition, and the physical and mechanical properties of the aggregate have an important effect on concrete shrinkage. The role of the aggregate is two fold:-

a. It reduces the shrinkage as the volume of the active component is reduced.

b. It produces a restraining effect on the volume changes.

The degree of restraint offered by the aggregate(87,89,90) is governed by the type of aggregate and its elastic properties. Carlson(91) concluded from his extensive research that the relative compressibility of aggregate particles appears to be the most important factor causing different aggregates to produce concrete of different shrinkage.

Prolonged continuous water curing has very little effect upon shrinkage. Lyse(92) came to the conclusion that the length of moist curing before placing in drying room had only minor effect on the final shrinkage of concrete. According to Fulton(93) moist curing can only defer shrinkage and not reduce it. The water/cement ratio, the properties of aggregate and cement are inter-dependent factors which in addition to curing conditions influence the shrinkage of concrete.

The action of carbon dioxide on hydrated cements leads to the formation of calcium carbonate and silica gel, a process that takes place at the low pressure of carbon dioxide in the normal environment, and this is invariably accompanied by shrinkage. It was shown by Verbeck(94) that shrinkage due to carbonation is found to be maximum at about 50% relative humidity. Lower carbonation shrinkage was observed both under lower and higher humidities owing to the lower rate of diffusion of carbon dioxide under these conditions. The above data indicates that shrinkage of laboratory specimens observed at a relative humidity of 50% appears to increase carbonation shrinkage.

2.7 DYNAMIC AND DAMPING CAPACITY OF CONCRETE.

The non-destructive method of testing laboratory specimens developed in America(95), was based on results which show that changes in concrete quality are related to changes in the dynamic (elastic) modulus deduced from measurements of the natural frequency of transverse vibration of concrete beams. The advantages claimed for this method of test are:-

1. The method is non-destructive so that in the study of the effects of factors such as ageing,frost damage, etc. the same specimen may be examined at progressive stages.
2. A higher order of reproducibility of results is obtained than with destructive static strength determinations and as a result fewer specimens are required to obtain a true average value.

3. The tests are simple and rapid in operation.

The dynamic modulus and damping capacity are the two important properties of engineering materials obtained by vibrational methods of testing. The greatest advantage of these tests, apart from their non-destructive nature is that the dynamic values are obtained from transient loads far below the elastic limit, so that the results are free from time dependent inelastic strains and are related to the internal structure of the material. For concrete, the dynamic properties are of great value in fatigue, creep and durability studies, in detecting the onset of deterioration due to freezing and thawing, and in investigations of the effects of weathering cycles. The damping properties of concrete are as important as dynamic modulus, e.g. in the design of tall buildings, it is important to consider the dynamic response to transient loading and natural frequency of the structure. To obtain high degrees of stability it is necessary to avoid the large amplitudes of vibration associated with resonant frequencies. A knowledge of both the dynamic modulus and the damping characteristics of cementitious materials is thus an important aid to design and can help in explaining the variation in their physical properties.

2.7.1 Determination of Modulus of Elasticity from Dynamic Tests.

The methods of obtaining the elastic modulus of solid bodies from their mechanical resonance frequencies originated(96,97) about forty years ago. Since then the propagation of vibrations in cementitious materials has been studied extensively, and the experimental techniques and methods of interpreting data have been developed(98,99,100). Exact solutions for cylindrical specimens, and approximate solutions for specimens of prismatic shape have been established to calculate the dynamic modulus of elasticity for the different modes of vibration(101). Design formulae and correction factors for the size and shape of the specimen and its Poisson's ratio are presented by Pickett(102) and Spinner and Teft(100) and have appeared in Codes(103), standards(104) and recommendations(105).

The relationship between elastic modulus and resonant frequency generally applies to homogeneous, isotropic and elastic solids, but may be applied to heterogeneous systems, such as concrete, when the dimensions of the specimens are large compared to the size of their constituents. With prismatic specimens the best results are obtained when the length-depth ratio is of the order of five(105).

The relationship between the dynamic modulus of elasticity and the longitudinal resonant frequency which forms the basis of the equation given in British Standards(103) was first derived by Rayleigh(106) and is given by Jones(107) as:-

$$E = \frac{\rho}{K_n} \left(\frac{2Lfr}{n}\right)^2 \qquad \ldots\ldots \qquad (2.3)$$

where

$$K_n = 1 - \frac{\pi n^2 \vartheta^2 d^2}{8\ L^2}$$

E = dynamic modulus of elasticity.

ρ = density of the material.

L = Length of the specimen.

d = Diameter of the specimen.

fr = Longitudinal resonance of frequency.

ν = Poisson's Ratio.

It has been(100) shown that equation (2.3) can be applied to prisms with a very high degree of accuracy. The term (K_n) for a concrete prism (100x100x500 mm) vibrating in its fundamental longitudinal mode is almost unity to within (± 0.002%) accuracy. Finally the equation was standardised and took the following form as seen in the British Standards(103).

$$E_D = 4\ fr^2\ L^2\ \rho \times 10^{-12} \quad \ldots\ldots \quad (2.4)$$

where E_D in N/mm^2, fr in hertz, L in mm, and ρ in kilogram/cu. m.

2.7.2 Determination of the Damping Capacity of Concrete.

The measurement of the damping capacity of a material has long been of interest because of its immediate practical value as a means of assessing quality. In recent years, the study of the variation of the damping capacity with change in such variables as temperature and frequency of vibration has led to much information about the internal structure of materials.

The energy per unit volume of the material dissipated internally per cycle of vibration, provides a measure of the damping present and is referred to as the damping capacity of the material. Therefore, damping capacity is the property of the material which causes vibrations in a specimen to decrease in amplitude even though the specimen is isolated in a vacuum to prevent energy loss through air resistance.

There are four(108) common ways of defining damping capacity mathematically; which one is selected depends largely upon the method of measurement employed. These methods are:

a. Direct method: to measure the decrease in amplitude of free vibrations and calculate the logarithmic decrement, δ, using the formula:

$$\delta = \frac{1}{n} \log_e \frac{A_o}{A_n} \quad \cdots\cdots \quad (2.5)$$

where A_o is an initial amplitude and A_n the amplitude after (n) cycles. For small amplitude of vibration, equation (2.5) may be expressed as:

$$\delta = \frac{\Delta A}{A} \quad \cdots\cdots \quad (2.6)$$

where ΔA represents the decay in amplitude per cycle.

b. In the case of forced vibrations of materials with small damping capacity, where the undamped natural frequency very nearly equals the damped frequency, equation (2.5) can be re-arranged as follows:

$$\delta = \frac{\pi}{\sqrt{r2-1}} \frac{f_2 - f_1}{fr} \quad \cdots\cdots \quad (2.7)$$

where fr = resonant frequency of vibration

f_1 and f_2 = frequencies on either side of resonance at which the amplitude of vibration is $\frac{1}{r}$ x the maximum. For 50% bandwidth, $r = 2$ and

$$\delta = \frac{\pi}{\sqrt{3}} \frac{f_2 - f_1}{fr} \quad \cdots\cdots \quad (2.8)$$

Equation (2.8) is based on the assumption that undamped and damped frequencies are the same, therefore the error involved in the resulting values of logarithmic decrement with materials such as concrete which exhibits low damping is of the order of 0.01%.

c. The magnification factor Q, which is often referred to as the damping constant, is sometimes used as a measure of the sharpness of the resonance curve. The relationship between Q and logarithmic decrement (δ) is given by:

$$\frac{1}{Q} = \frac{\delta}{\pi} = \tan^{-1}\phi \qquad \cdots\cdots \qquad (2.9)$$

where (ϕ) is the phase angle which represents the lag of strain behind the applied stress in forced oscillations of a specimen.

d. The specific damping capacity (S) is defined as the ratio of the strain energy lost per cycle to the total strain energy in the specimen in that cycle. The value of (S) is given by:-

$$S = \frac{\Delta E}{E} \qquad \cdots\cdots \qquad (2.10)$$

where ΔE is the energy in each cycle which is transformed into heat or internal strain energy and E is the strain energy at the maximum amplitude.

2.8 BOND STRENGTH.

2.8.1 Nature of Bond.

Bond is defined(109) as"adhesion of concrete or mortar to reinforcement or to other surfaces against which it is placed" and bond strength is defined as "resistance to separation of mortar and cement from reinforcing steel and other material with which it is in contact". Bond also has been defined as "that property which causes hardened concrete to grip an embedded steel bar in such a manner as to resist forces tending to slide the bar longitudinally through the concrete".

Bond between concrete and steel, or the resistance to withdrawal of steel embedded in concrete, may be divided into three elements(109,110):

1. Adhesion or glueing by the cement gel.
2. Friction between bar and concrete caused by lateral internal forces.

3. Keying between the concrete and the bar deformations which depends on the bearing and shearing resistance of concrete.

In the adhesion stage, the two materials deform together without relative movement. Several theories have been put forward to explain the term adhesion and of these, that of micromechanical locking appears to be most reasonable. The theory postulates that resistance to slip is a measure of the shear strength of the fine particles of concrete forced into the micro-indentations on the surface of the steel and that the adhesive resistance ceases to act when an initial slip comparable to that of the size of the micro-indentation takes place. Pogany(111) believes that in the process of strength development, the gel formation and subsequent crystallization fill all the irregularities of the steel surfaces and this causes roughening of the steel surface and thereby increases the adhesive area. Thus it becomes quite evident in the light of the above theory that the adhesive resistance is mainly dependent on the surface roughness of steel reinforcement.

In the adhesion stage it is assumed(109) that, as a load is applied to a bar embedded in concrete, the two materials will strain together elastically and that this liaison will end when the deformation of the concrete reaches a certain critical value. From this assumption, the movement of a point such as (A) shown in fig. (2.3), relative to the mass of the concrete will vary directly with the value of applied load. When the subjected strain of the concrete adjacent to the steel reaches a critical value, the adhesion will be ruptured at point "A".

As the applied load is continued, rupture of the adhesion will occur over a length of the bar and relative movement between the steel and the concrete will have taken place over this length. The junction between the adhesive and frictional stages which will be called the rupture point(109), will have moved along the bar away from the loaded end.

In the frictional stage, the steel bar moves relative to the concrete and experiences a frictional resistance which is based on the interlocking of the uneven, rough bar surface with the mortar. The frictional resistance is also dependent on the uniformity and magnitude of the lateral pressures developed between the steel and the surrounding concrete. Thus the bond strength, at least in the initial stages of slip, may depend less on the rigidity of the hardened cement paste than on the tendency of the concrete to shrink away from the steel during the process of setting. It is also conditioned by the volumetric fluctuations of the surrounding concrete with changes of moisture content and temperature.

In the case of deformed bars, an additional resistance to slipping is brought about by "lug action" or the bearing of the lugs on the concrete. The ribs or projections in the bar press against the concrete in contact with them once sufficient movement between the steel and concrete has developed. The bond resistance at ultimate stress will consist of friction between the bar and the concrete and the bearing forces against the lugs. The bearing pressure(112) developed at the lug interface can be resolved into two components as shown in fig. (2.4), one parallel to the bar tending to shear a

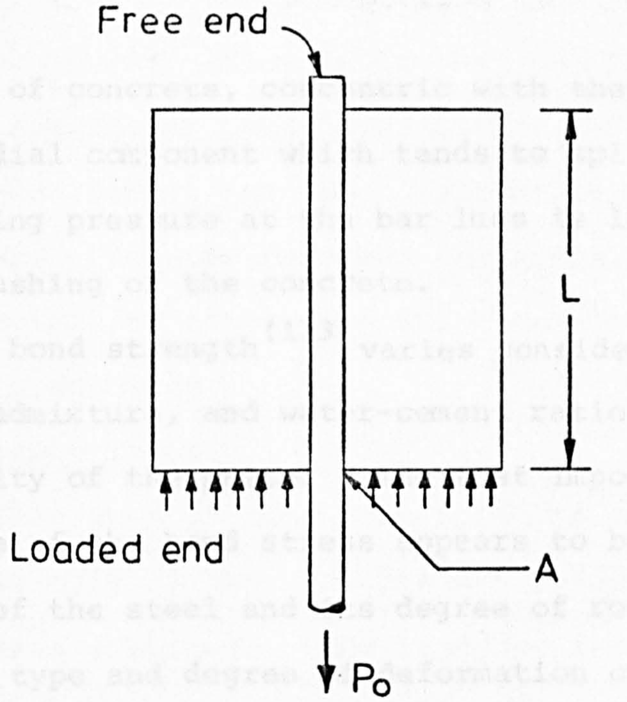


FIG. 2·3 DETAILS FOR THE PULL-OUT TEST SPECIMEN

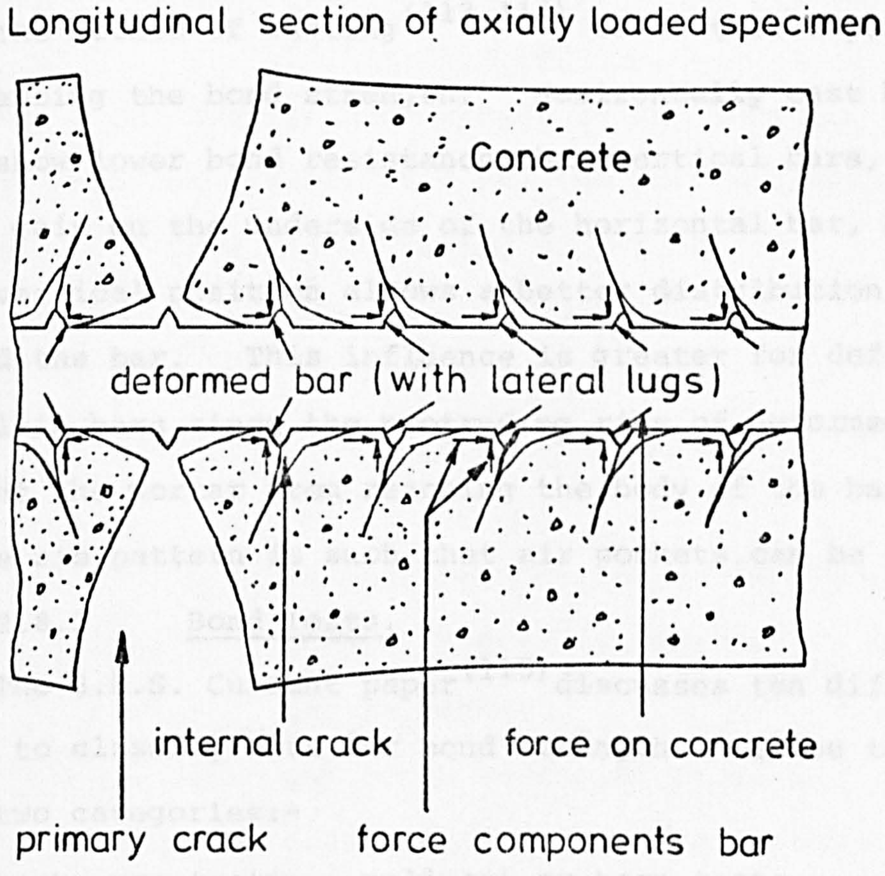


FIG. 2·4 DEFORMATION OF CONCRETE AROUND REINFORCING BARS cylinder of concrete, concentric with the bar deformations, and a radial component which tends to split the concrete. If the bearing pressure at the bar lugs is large, it can cause local crushing of the concrete.

The bond strength(113) varies considerably with the types of cement, admixture, and water-cement ratio, all of which influence the quality of the paste. The most important influence on the magnitude of the bond stress appears to be the character of the surface of the steel and its degree of roughness. It varies with the type and degree of deformation of the bar surface, the bar diameter, and the cover of concrete. The magnitude of bond strength increases with delayed vibration and it is greater for dry concrete than for wet concrete.

The method of casting(113,114) is another important factor influencing the bond strength. Horizontally cast bars generally show lower bond resistance than vertical bars, because of water gain on the underside of the horizontal bar, and casting in a vertical position allows a better distribution of mortar around the bar. This influence is greater for deformed than for plain bars since the protruding ribs of deformed bars may prevent the mortar from reaching the body of the bar, particularly if the rib pattern is such that air pockets can be formed.

2.8.2 Bond Tests.

The B.R.S. Current paper(115) discusses ten different bond tests to classify bars for bond strength. These tests fall into two categories:-

1. Anchorage tests:- pull-out or beam tests.

In this type of test, the force applied at one section of the bar is transferred progressively to the concrete. These tests give an indication as to when the bond stress exceeds the bond strength. They also give a relationship between the average bond stress and the displacements measured at the end of the bar. In some European countries, e.g., England and France, spiral or steel ties are recommended to avoid splitting failure in concrete.

2. Transfer Tests:-

In this type of test, the force is applied at the two protruding ends of the bar, the bar being continuous through the concrete specimen. The transfer test produces tension cracks in the specimen with increasing steel stresses. This test represents more realistically what happens in the central tension zone of a concrete beam.

2.8.3 Bond Stress Distribution and the Techniques Used in its Measurements.

The local average bond stress at any location on the interface between the embedded bar and the surrounding concrete can be found(116) from the following equations:

$$U = \frac{dT}{\varepsilon_o \, dx} \quad \ldots\ldots \qquad (2.11)$$

or

$$U = \frac{A_s \, dfs}{\varepsilon_o \, dx} \quad \ldots\ldots \qquad (2.12)$$

where U = average bond stress at the location considered.

dT = change in force in the bar in length dx.

ε_o = bar perimeter.

A_s = bar cross sectional area.

dfs = change in steel stress in the bar in length dx.

It is clear that the bond stress is directly proportional to the shape of dfs/dx of the steel stress distribution curve, but equation (2.12) can be restated in terms of the steel strain (ε_s) and the elastic modulus for the steel (E_s) as follows:

$$U = \frac{A_s E_s}{\varepsilon_o} \frac{\partial \varepsilon_s}{\partial x} \qquad \ldots\ldots \qquad (2.13)$$

in which ε_s is the strain in the steel bar at the location considered.

From equation (2.13), it is clear that the bond stress at any location along the interface, at any load, is proportional to the slope of the steel strain distribution curve at that point for the given load. Bond stress found in this way can also be plotted as a continuous function of distance along the bar.

The steel strain distribution along the embedded length of the bar has been found using different techniques. Some investigators, for instance, have used electric resistance strain gauges bonded on the outer surface of the steel bar. This however is objectionable because the presence of properly water-proofed gauges at the desired close spacing modifies the distribution of stress and force transfer across the concrete-steel interface.

Watstein(117), on the other hand, measured the strains in the steel at three locations equally spaced at 2-inch centres by means of mechanical gauges through holes in the concrete. Stresses in the steel bar at these points were calculated from the observed strains and the bond stress distribution was obtained by calculating the rate of the steel stress variation along the bar length. It was found that the bond stress along the bar was considerably more uniform for the short length of embedment than for the longer ones. Within the available precision, Watstein concluded that, for deformed bars, the bond stress at the loaded end increased with the increase in the applied pull-out force. For smooth bars, his conclusions were similar to those of Glanville(118), who showed from tests on smooth tubes that the maximum bond stress shifted progressively from the loaded end to the free end as the applied pull-out force was increased.

Mains(119) used a new technique to study the bond stress distribution in beams and pull-out specimens. The steel bar was split longitudinally into two parts, one of them was a thin slice. The larger slice was then milled to provide a channel (0.31 in. wide by 0.175 in. deep) for gauges and lead wires. The channel was cut so as to allow the gauges to be mounted as near the centroid of the finished bar section as possible without local bending of the bar. This technique furnished a much more detailed picture of undisturbed bond stress distribution. A variation of this method was used by Johnston(120) who employed two slots on the surface of the bar cut along the axis of the bar, each diametrically opposite to the other. The strain gauges were mounted in both slots and the slots were eventually filled with water-proofing material.

Mains reported(119) that the local maximum values exceeded the average bond stress values by a factor of 2 or more. Thus, the usual reinforcement anchorage requirements, which treat the bond as a constant level of stress along the bar, are incorrect. In this test, the progressive displacement of the peak bond stress from the loaded end to the free end for plain bars with an increase in the applied loads, was confirmed. With deformed bars, the maximum bond stress was found to be near, or at the loaded end for low applied steel stresses, but with a further increase of the applied load up to the yielding of the bar, the maximum bond stress was found to move progressively away from the loaded end by about 10-30% of the embedded length.

Perry and Thompson(121) tested 3 eccentric pull-out prisms reinforced with commercially available standard deformed No. 7 hard grade steel bars. The steel stresses were measured using Mains technique but with some modifications. Two identical reinforcing bars were used, each bar was sawn longitudinally in half and ⅜ in x ⅛ in. groove was milled out of the flat surface of each half bar. Electric resistance strain gauges were then mounted in the grooves of each half bar. All gauges were water proofed and then the two halves of each bar were welded together using tack welds at several points along the bar. They concluded from their measurements that increases in the bar loads caused more end slips and caused the point of maximum bond stress to propagate towards the free end, just ahead of the splitting crack observed in the thin concrete cover.

Wilkins(113) measured bond stress distribution along a smooth tube embedded in a concrete cylinder and used as a pull-out specimen. The gauges were inserted in the tube by means of brass wedges designed to fit the bore of the tube and operated by steel rods from each end of the tube. The experimental curves predicted by this method show clearly that at the loaded end the friction bond is much less than at the free end where it has practically maintained its maximum value. He also found that maximum bond stress moves towards the free end, and the first slip at the free end is observed where the bond stress at that end reaches its limiting value.

Rehm(122) conducted pull-out tests of specially made bars having a single projecting annular rib representing one deformation, and having a very small length of embedment. He developed what is termed as a "fundamental law of bond" in exponential form. He appears to be the first investigator to attempt to develop a force-deformation relation for bond on a localized basis. Nilson(116) used an internal strain gauge to determine the slip between steel and concrete at internal locations along the interface. Using these gauges it was possible to measure the concrete strain, close to an embedded bar, at discrete locations along the length of test specimens. When both the steel and concrete strain distributions are known, the relative displacement between the steel and concrete may be found, from which slip at any location is easily determined. Ruiz(123), as reported by Nilson(116), developed a slip gauge using a standard electrical resistance strain gauge installed with the central portion unbonded. One end of the gauge was bonded on the reinforcing bar, while the other end was bonded to a small cube of mortar embedded later in the concrete.

In the present work, the specimens were dried at 105° C for two weeks and then soaked in water or crude oil for a further period to ensure a high degree of saturation before testing. This required the use of a special technique to minimize the effect of drying and absorption of both water and oil on the measured bond stress along the embedded bar. Main's technique(119) was, therefore, found to be satisfactory and adopted for the experimental work as described in detail in section (5.3.2).

CHAPTER 3.

EXPERIMENTAL PROGRAMME AND MATERIALS.

3.1 INTRODUCTION.

Concrete is an extremely variable and highly complex composite material; its properties depend on a number of factors such as the properties of the mix constituents, casting and curing conditions. Thus, to have a clear idea about the effect of oil on the studied properties, all other factors which might affect the concrete characteristics were kept constant, as far as possible, throughout the test.

3.2 EXPERIMENTAL PROGRAMME.

The primary aim of this investigation was to determine the variation in physical properties of concrete saturated with crude oil, in order to control the problem associated with oil soaked concrete encountered in many industrial processes such as concrete storage tanks, machine foundations, garage floors and oil drilling rigs.

The various experimental programmes are detailed in the relevant chapters but the main areas of study are:-

1. The effect of oil on the strength of cement paste, mortar and concrete (compressive and tensile) after being soaked for different periods i.e. containing different amounts of absorbed oil, and the variation of this effect with w/c ratio, size of aggregate, size of specimen and different curing conditions. A full degree of saturation with oil was required to determine the variation in the measured properties. The

FIELD
UNIVERSITY
LIBRARY

experimental work includes the measurement of the volumetric strain,Poisson's ratio, moisture movement, modulus of elasticity and the degree of saturation with oil for different periods up to 18 months.

2. The behaviour of concrete and the degree of deterioration when soaked in crude oil compared to that soaked in water or kept in air by measuring the dynamic modulus of elasticity and damping capacity.

3. The variation in the bond strength between concrete and steel due to soaking in oil for different periods compared with two other curing conditions. In the pull out tests, two concrete mixes and two types of reinforcing bars (plain and deformed) were used. A modified pull out test was used to measure the bond stress distribution along a plain bar with electrical resistance strain gauges using 18 concrete specimens with two w/c ratios and three different curing conditions.

3.3 MATERIALS.

3.3.1 Cement.

A standard brand of ordinary Portland cement (OPC) manufactured by the Blue Circle Group was used in all mixes employed in this work. Its approximate chemical compositions are given in table (3.1).

3.3.2 Aggregate.

Natural river aggregates were used throughout this investigation. The coarse aggregate was continuously graded irregular shaped gravel obtained from the River Trent, Derbyshire. Two types of aggregate grading, one with maximum size of 10 mm and the other with maximum size of 20 mm, were used in this investigation.

Compound Composition	Abbreviation	% by weight
Tricalcium silicate	C_3 S	63.1
Dicalcium silicate	C_2 S	8.9
Tricalcium aluminate	C_3 A	11.0
Tetracalcium aluminoferite	C_4 AF	6.4
Silica	SiO_2	19.7
Insoluble residue		1.5
Alumina	Al_2O_3	5.5
Ferric oxide	Fe_2O_3	2.1
Lime	CaO	65.5
Magnesium oxide	MgO	1.1
Sulphur trioxide	SO_3	2.5
Alkalis	Na_2O	0.2
	K_2O	0.6
Loss on Ignition	I.	1.4
Specific surface		333 m^2/Kg
Specific gravity	S.G.	3.12

Table (3.1) Chemical composition of the Ordinary Portland Cement (OPC) used in the investigation.

The 10 mm maximum size aggregate was used because of its common use in high quality concrete for a heavily reinforced section and particularly prestressed concrete members. As Neville(124) pointed out, the maximum size of aggregate should be between 1/5 to 1/4 of the structural concrete section, and since it was necessary in this investigation to make small specimens in order to get the concrete quickly saturated with oil, 10 mm aggregate was required. A 20 mm maximum size aggregate was used to compare with the coarse aggregate of 10 mm and to study their effect on the mechanical properties and the degree of oil saturation. The fine aggregate was a dried river quartzite sand graded to zone 2 of B.S. 882:1201:1973(125). They were obtained from the River Trent,Derbyshire.

Grading, fineness modulus and water absorption tests carried out for both dried sand and the coarse aggregate are shown in tables (3.2) and (3.3) and the grading curves obtained from the sieve analysis are plotted in Figs. (3.1) and (3.2).

Fine Aggregate Properties.

Washed and dried river sand
Zone 2 grading
Water absorption 0.86%
Fineness modulus = 2.646.

Grading:

Sieve Sizes in	Sieve Sizes mm	% Passing	B.S. Requirement
3/8	9.52	100	100
3/16	4.72	98.9	90 - 100
No. 7	2.40	87.3	75 - 100
14	1.20	76.5	55 - 90
25	0.60	52.1	35 - 90
52	0.30	16.6	8 - 30
100	0.15	4.0	0 - 10

Table (3.2) Fine Aggregate Grading.

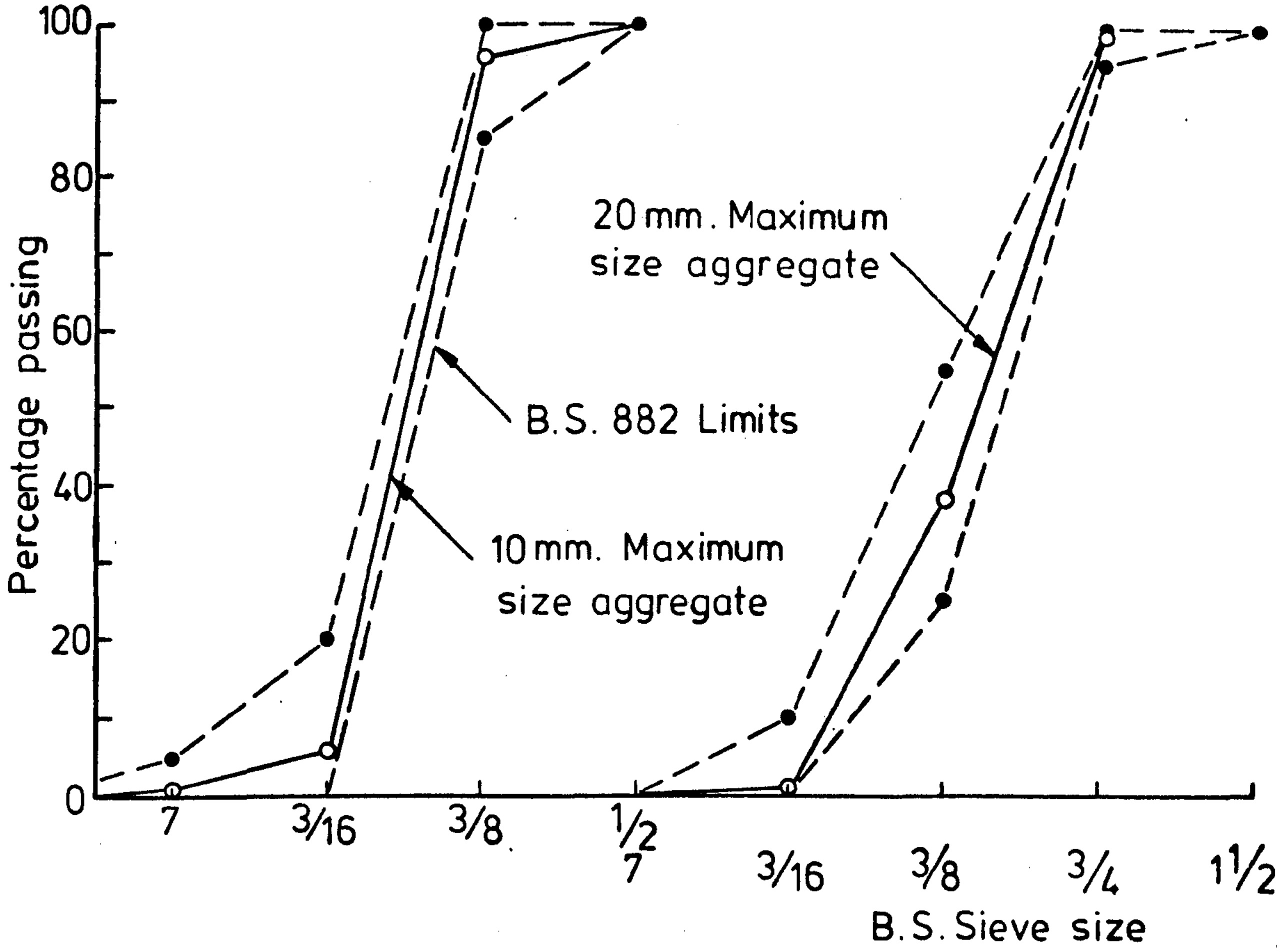


FIG. 3·1 GRADING CURVES FOR 10 AND 20 mm COARSE AGGREGATES

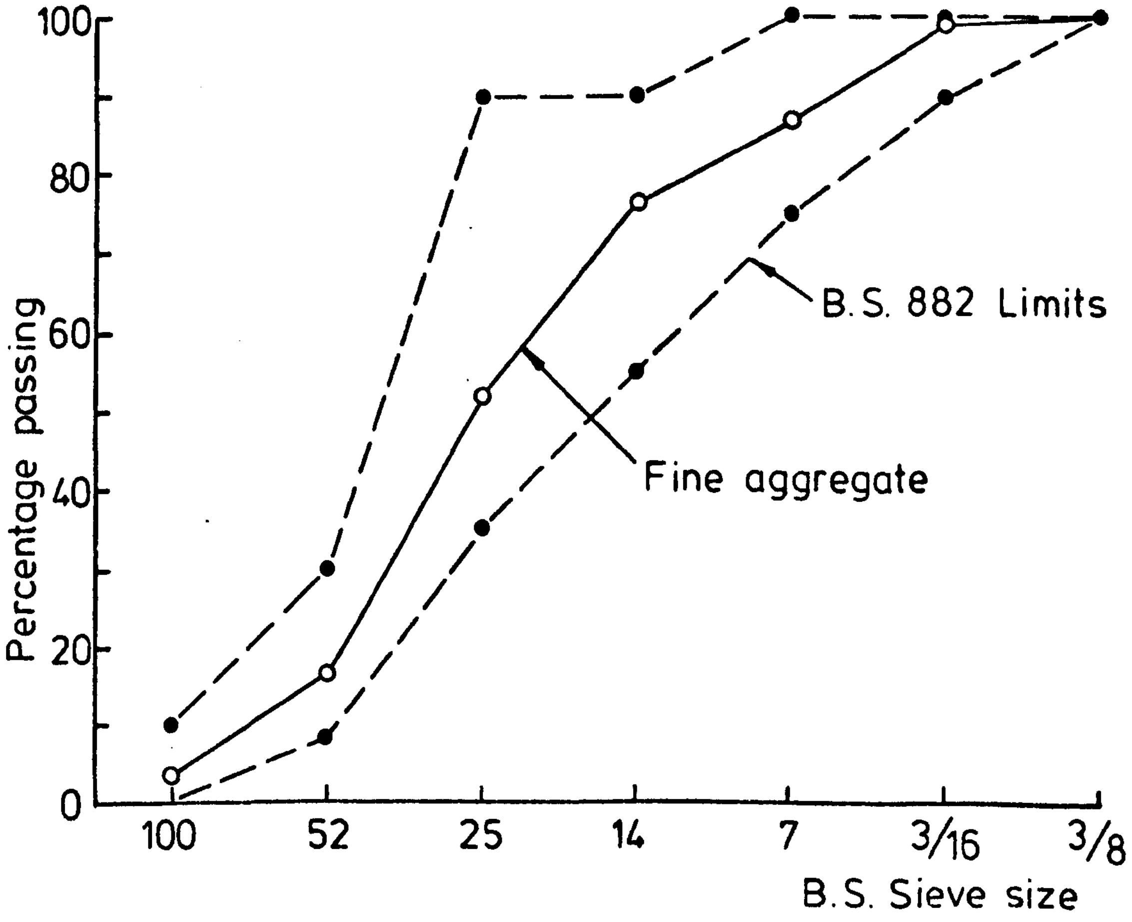


FIG. 3·2 GRADING CURVES FOR FINE AGGREGATE

Coarse Aggregate Properties.

Natural and irregular river gravel.
10 mm (⅜") maximum size.
Water absorption 0.9-1.2%
Fineness modulus = 5.97.

Grading:

Sieve Sizes		% Passing	B.S. Requirements
in	mm		
3/4	19.0	100	100
1/2	12.7	100	100
3/8	9.52	96	85 - 100
3/16	4.72	6	0 - 20
No. 7	2.40	0.8	0 - 5

Natural and irregular river gravel.
19.0 mm (¾") maximum size.
Fineness modulus = 6.60
Water absorption:- 0.5 - 0.72%

Grading:

Sieve Sizes		% Passing	B.S. Requirements
in	mm		
1.1/2	38.00	100	100
3/4	19.00	100	95 - 100
3/8	9.52	39	25 - 55
3/16	4.72	1	0 - 10
No.7	2.40	0	-

Table (3.3) Coarse Aggregate Grading and Properties.

3.3.3 Steel.

The reinforcing steel used in the bond specimens was 20 mm diameter and of two types, high stress - high bond (deformed bars) and plain round mild steel bars. The bar used in the modified pull out test to measure the bond stress distribution was low-carbon, free cutting (E.N.1.A.) plain round mild steel bar.

The properties of the bars were determined from tensile tests. Typical stress-strain curves are shown in Fig.(3.3). The strain was measured over a gauge length of 50 mm using an extensometer placed at the central portion of the tested bar.

3.3.4 Crude Oil.

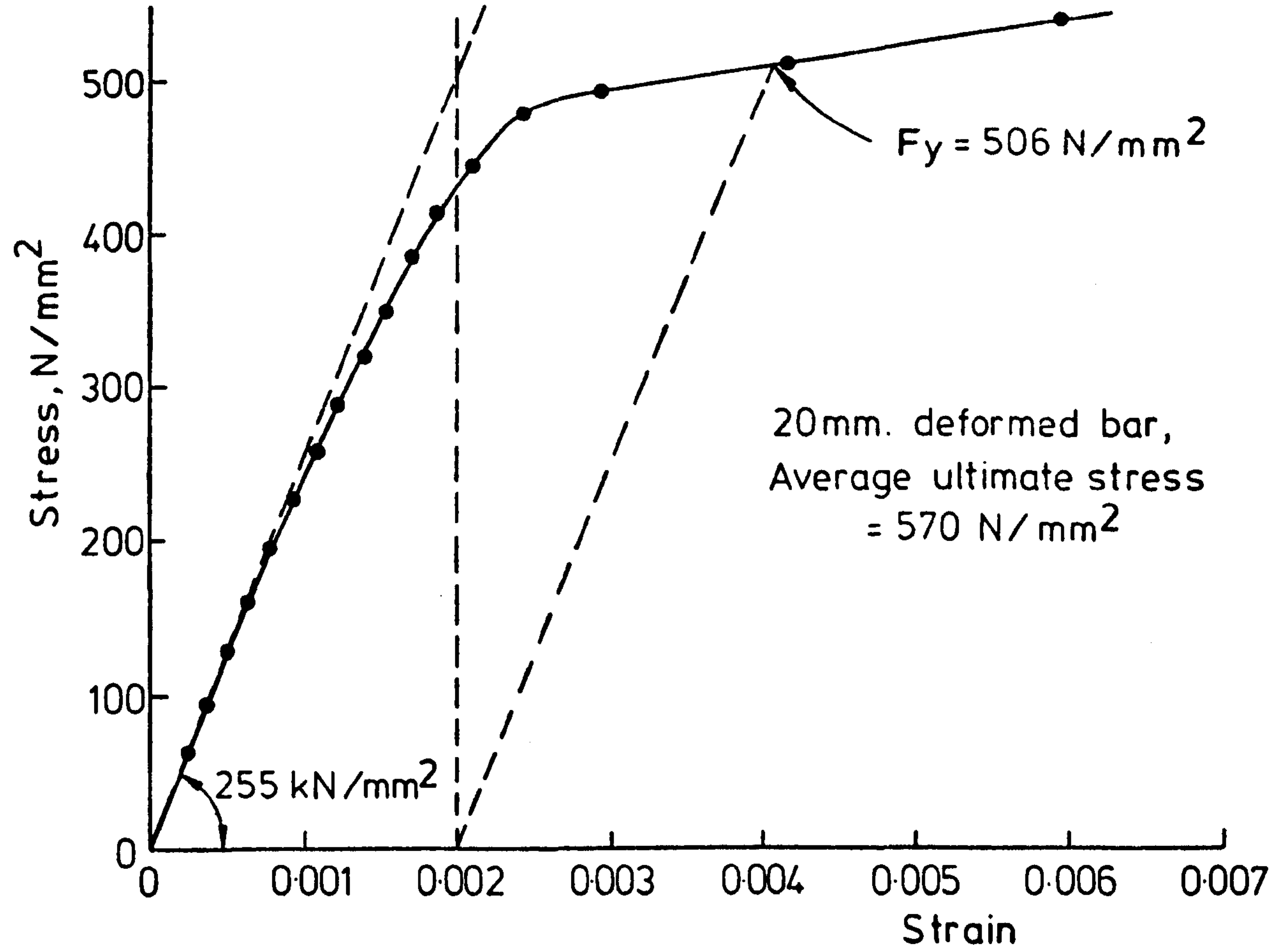
Kuwait crude oil was used throughout the test. A chemical analysis was carried out by the chemistry department of this university to identify its chemical composition, table (3.4). Earlier tests(18,20) had established that the main properties of the crude oil do not change significantly after contact with concrete over prolonged periods.

1. Distillation up to 100° C (Cumulative % of total volume)

Temperature °C	% Volume (cumul.)
40 - 80	4.70
80 - 90	9.12
90 - 110	10.22
110 - 120	11.88
120 - 145	14.64
145 - 159	18.51
159 - 165	20.72
165 - 181	34.26
181 - 190	43.10
190 - 200	47.24
200 - 218	59.12
218 - 235	60.03
235 - 255	75.70
255 - 280	85.37
280 - 290	83.96
290 - 295	92.27
295 - 300	100.00

2. Sulphur content 2.07 - 2.09%
3. Moisture Content (as H_2O) 1.15%
4. Calcium Content non detected
5. Specific gravity (at 16° C) 0.890 - 0.891
6. Viscosity at 15° C. 0.2243 centistrokes
   30° C. 0.2045 centistrokes
   60° C. 0.1847 centistokes.
7. Wax content 5.7% by wt.
8. Surface tension at 16° C. 32.17 dynes/cm
9. Molecular sizes highly variable ranges 1 to 18 Å
   high occurrence of between 5 and 7 Å.

Table (3.4) Analysis of crude oil.



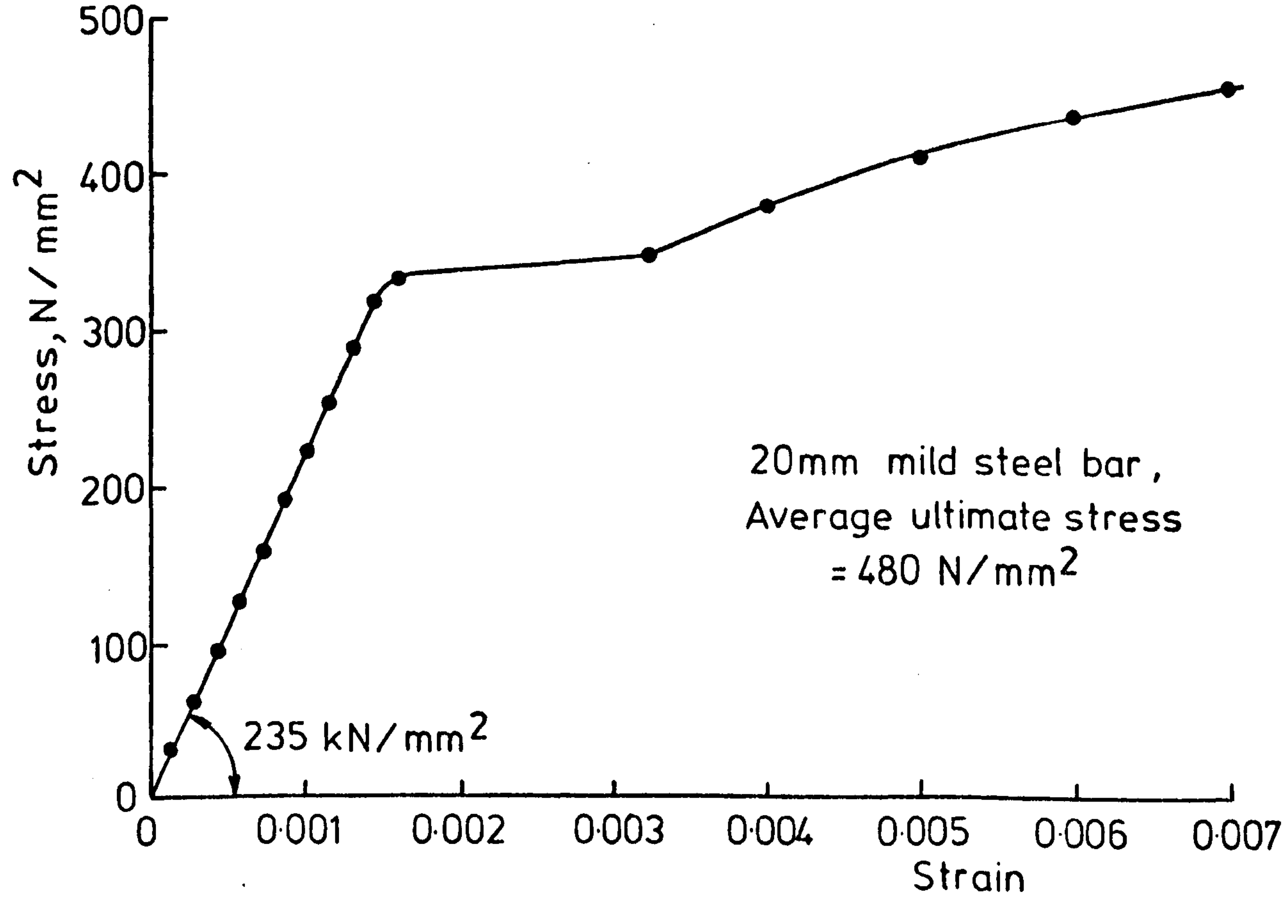


FIG. 3·3 TYPICAL STRESS – STRAIN CURVES FOR THE BARS USED IN THE PULL-OUT TEST SPECIMENS

3.4 PREPARATION OF CEMENT PASTE AND MORTAR SPECIMENS.

3.4.1 Casting.

Only one type of cement (OPC) was used in all tests three water cement ratios (0.3, 0.35, 0.4) by weight, were chosen to cover a maximum range of usable cement paste. For the mortar, two proportions of cement to sand were chosen. Proportion 1:2 covers the range of water cement ratios (0.4, 0.5, 0.6) while proportion 1:3 covers the range of water cement ratios (0.5, 0.6, 0.7). Both proportions include the water cement ratio of 0.5, so that a cross check can be obtained. The sand used in the mortar was the same type as that used in concrete but graded to pass No. 14 Sieve and retain on No. 100 Sieve.

The cement paste or mortar used in this investigation was mixed in a bowl type laboratory Hobart bench mixer - plate (3.1). The cement paste or mortar was mixed in the following manner.

1. Place all the mix water in the bowl.
2. Add cement and mix for 30 sec. at slow speed (185 r.p.m.). In case of mortar, add the entire quantity of sand slowly over another 30 sec. period, while mixing at slow speed.
3. Stop the mixer, change to medium speed (285 r.p.m.) and mix for 30 sec.
4. Stop the mixer, let the cement paste or mortar stand for 1½ min. Scrape down into the batch any cement paste stuck to the sides within the first 15 secs.
5. Finish by mixing for 1½ min. at medium speed.
6. For cement paste of w/c = 0.4 and mortar of w/c = 0.7, remixing for another period of 1½ min. is required to increase the effectiveness of mixing.

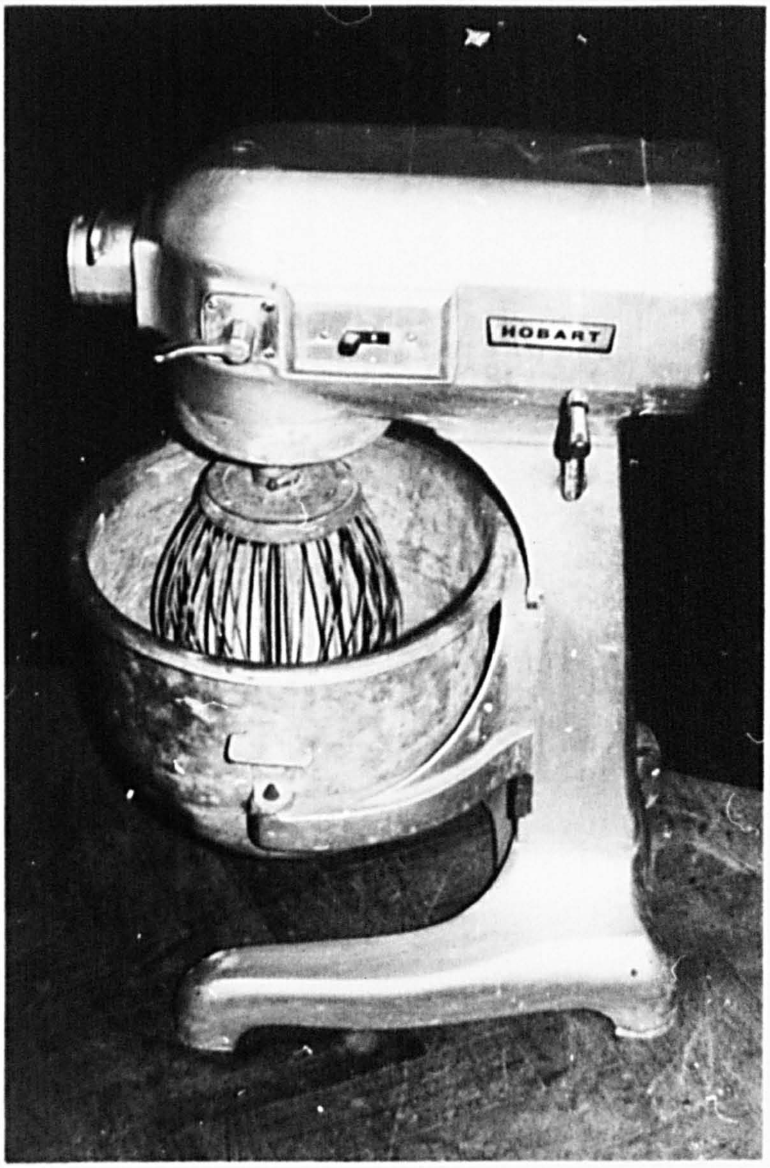


PLATE. 3-1. HOBART BENCH MIXER

After mixing, the cement paste or mortar was poured into moulds in two layers, each layer being vibrated for about 1 minute. The tops were trowelled flat after about 2 hours and covered with a polythene sheet for about 24 hours, after which the specimens were demoulded and stored in water at a constant temperature of 16 ± 1° C for 7 days.

3.4.2 Sizes and shapes of the H.C.P. and Mortar Specimens.

The specimens were made as small as possible in order to get them saturated with oil in a short time. The specimens used for various investigations were:-

1. Compressive strength - 2" (50.8 mm) cubes.
2. Direct tensile strength - Briquet specimens as shown in plates (3.2) according to ASTM (C190-72).(126)
3. Modulus of Elasticity - 50x50x150 mm prisms, which provide a length to width ratio of 3 which is the same as for the concrete specimens with a gauge length of 50 mm.

3.5 PREPARATION OF CONCRETE SPECIMENS.

3.5.1 Concrete Mixes.

The water-cement ratio was considered to be the principle factor governing the mix design for all the mixes in this investigation. Mix design charts by Teychenne et al.(127) were used as a guide for obtaining the mix proportions for both trial and final mixes. Two types of aggregate grading were used, one with a maximum size of 10 mm and the other with a maximum size of 20 mm. For each type of aggregate, three w/c ratios were considered namely 0.4, 0.5 and 0.6 by weight. The mixes were designed to have a medium workability of 30-60 mm which is suitable for heavily reinforced sections using vibration. The aggregate/cement ratio was obtained for the corresponding water/cement ratio, medium workability, maximum size of irregular gravel and sand zone 2. The mixes used for both types of aggregates are given in table (3.5).

Max.Size of Aggregate	Mix Design	w/c Ratio	Cement Content (Kg/m^3)	Proportion by weight		
				Cement	Sand	Gravel
10 mm	A1	0.4	575	1.0	1.22	1.50
10 mm	A2	0.5	460	1.0	1.68	1.98
10 mm	A3	0.6	383	1.0	2.24	2.44
20 mm	B1	0.4	525	1.0	1.12	2.12
20 mm	B2	0.5	420	1.0	1.55	2.65
20 mm	B3	0.6	350	1.0	2.10	3.25

Table (3.5) Details of concrete mixes used throughout this investigation.

3.5.2 Casting of concrete specimens.

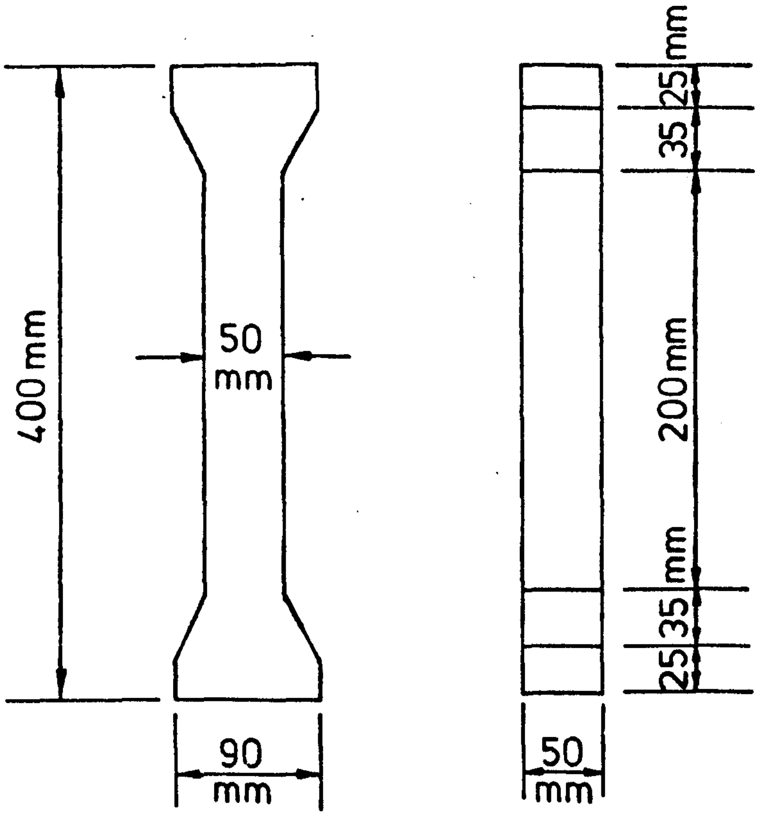
The sand was generally dried to laboratory temperature (about 20° C) before use and then the moisture content and absorption having been established, the weight of mixing water was adjusted to bring the sand to a saturated surface (SSD) condition prior to mixing. The coarse aggregate was soaked in the mixing water in a sealed tin for 24 hours before mixing to bring the aggregate to a saturated condition. The water absorbed by the coarse aggregate during the 24 hour soaking period was calculated according to B.S.812:1967, and added to the mixing water prior to mixing. Batching was by weight to the nearest 1 gm. Mixing was done in a $1m^3$ capacity horizontal pan type mixer with the coarse aggregate and the mixing water put first and the sand and cement then added. The constituents were mixed wet for 3 minutes and the slump was measured immediately after mixing.

The mixed concrete was poured into the cast iron moulds in 2 layers. Prior to use, the moulds were coated with a thin layer of mineral oil, which served to aid in the release of the samples from the moulds after hardening. Each layer of concrete being internally vibrated using 12 mm (½") poker vibrator until no air bubbles emerged from the surface of concrete but not long enough to cause segregation. The faces were trowelled flat after 2 hours and covered with polythene sheets in the open air laboratory, then demoulded after 20-24 hours.

3.5.3 Shapes and Sizes of Concrete Specimens.

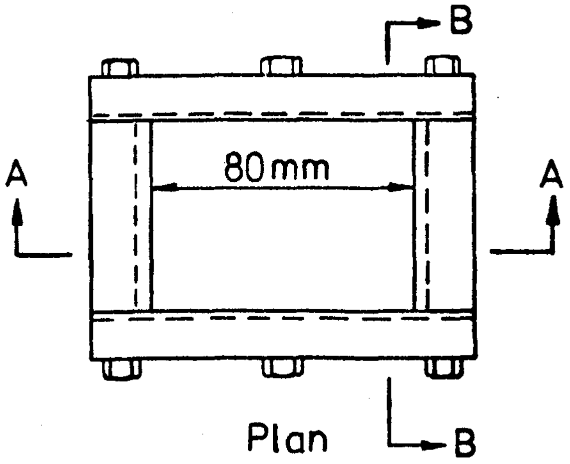
The size and shape of the specimens was governed by the ease of fabrication, handling and storage in addition to the recommended standards and test requirements. Smaller specimens were chosen to ensure a high degree of oil saturation in a shorter time. Accordingly, the following specimens were used for,

1. The Strength of Concrete
    a. Compressive tests: 100 mm cubes, 100 x 200 mm cylinders, 100x100x300 mm prisms.
    b. Tensile tests: 100x200 mm cylinder (indirect tensile tests), 50x50x400 mm briquet (direct tensile test as shown in fig. (3.4) and plate (3.3).

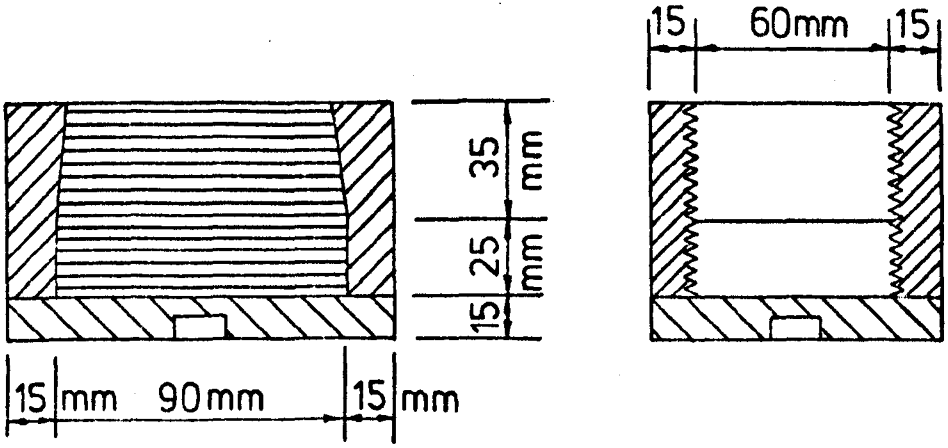


Elevation End elevation

FIG. 3·4A DETAILS OF TENSILE SPECIMEN



Plan



Section AA Section BB

FIG. 3·4B DETAILS OF END CAP

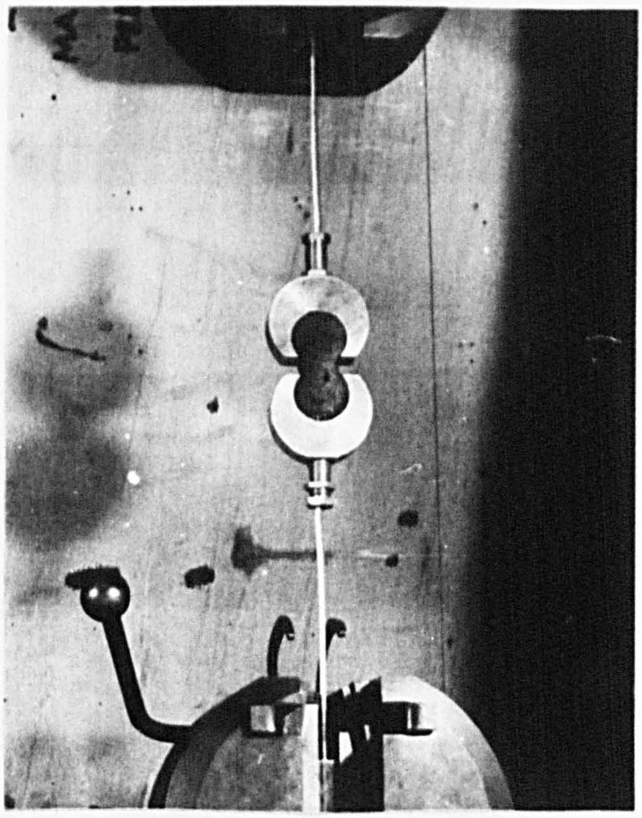


PLATE. 3-2. TESTING A BRIQUET FOR DIRECT TENSILE STRENGTH OF H.C.P. AND MORTAR

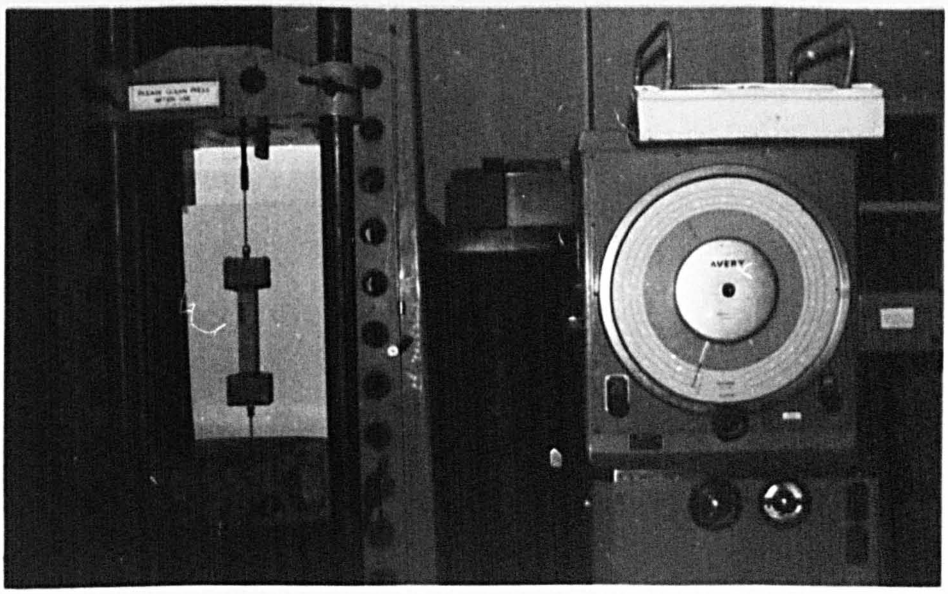


PLATE.3-3. DIRECT TENSILE TEST FOR CONCRETE SPECIMEN

    c. Modulus of rupture: 100x100x500 mm prisms.

2. Moisture Movement: 100x100x300 mm prisms.
3. Dynamic Properties: 100x100x500 mm prisms.
4. Bond Strength Tests:
    a. pull out test. 150x150 mm cylinder.
    b. transfer test (modified pull out). 100x100x500 prisms.

3.6 CURING.

After demoulding, the concrete specimens were stored in water tanks in the laboratory at 20 ± 3° C for 7 days. Then they were dried at a temperature of 105° C in a mechanically ventilated oven to a constant weight or until the weight loss was about 1 gm/day. This normally took 14-18 days. The oven temperatures were raised or lowered at a rate not greater than 8° C/hour to avoid thermal cracking of the specimens. After drying, the specimens were allowed to cool to room temperature. In order to study the effect of saturation with oil, the specimens were divided into three groups as follows:-

1. Reference specimens completely sealed in polythene bags and kept in the laboratory at a temperature varying between 16 - 20° C.
2. Specimens soaked in water tanks in the laboratory at the same temperature as the sealed specimen.
3. Specimens soaked in a shallow tank of crude oil at atmospheric pressure and at the same temperature as the sealed specimens.

Other specimens were kept continuously saturated in water at normal temperature to study the effect of drying and resoaking on the measured properties. Thirty days before testing, the oil soaked specimens were placed in a pressure vessel, plate (3.4), containing the same crude oil but under a pressure of 0.827 N/mm^2 (120 psi).

Hardened cement pastes and mortars were similarly treated except that all were oven-dried to 105^o C with the samples completely wrapped with aluminium foil to ensure a more uniform temperature in the specimen before any water is allowed to dry out. When the specimens reached the uniform temperature of 105^o C after about 24 hours, they were quickly unwrapped inside the oven and left to dry out to a constant weight for another 3 days.

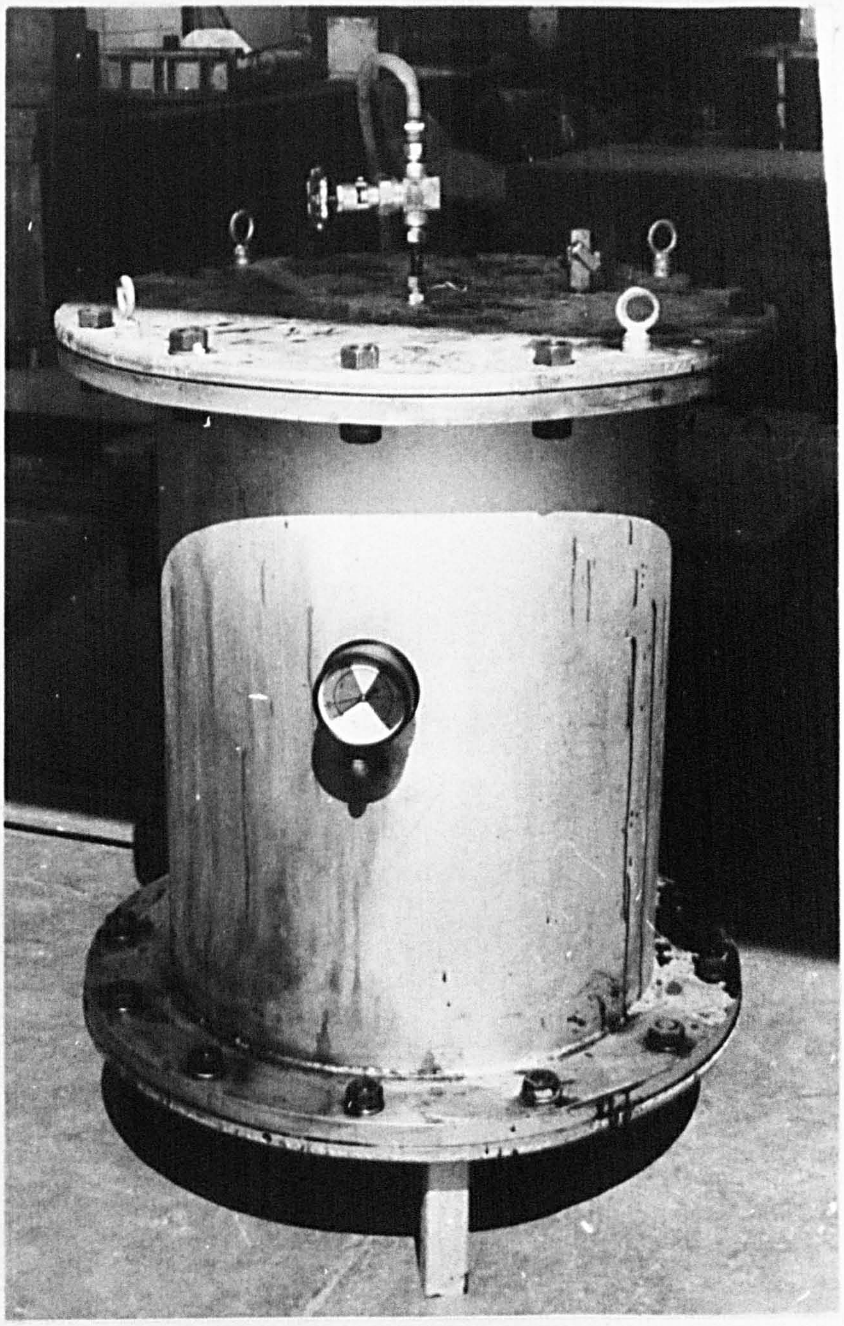


PLATE. 3-4. CRUDE OIL PRESSURE VESSEL

CHAPTER 4.

SOME MECHANICAL PROPERTIES OF OIL SATURATED CEMENT PASTE, MORTAR, AND CONCRETE.

4.1 INTRODUCTION.

In this chapter, the primary aim was to determine the variation in physical properties of concrete saturated with crude oil.The compressive strength of cement paste, mortar and concrete was investigated at various ages and under different curing conditions. Properties other than compressive strength, such as, tensile strength, modulus of elasticity, Poisson's ratio, pulse-velocity and moisture movement are also being investigated. The results show the changes that occur in these properties as a result of crude oil penetration. Oil saturation of the specimens was produced according to the procedure described in section (3.6). Table (4.1) shows the tests done throughout the investigation for each mix design.

4.2 EXPERIMENTAL PROGRAMME AND PROCEDURE.

4.2.1 Cement Pastes and Mortars.

The specimens were cast and cured as described in the previous chapter (Sections 3.4, 3.5, 3.6). The compressive strength cubes were tested at a loading rate of 15 N/mm^2 per min. Samples kept in oil were cleaned thoroughly to remove excess oil before testing so that friction between the cubes and the platens would be unaffected.

The direct tensile samples were briquet specimens according to ASTM (C190-72)$^{(126)}$. The specimens were held in the testing machine by a special clip fixed to a flexible cable to avoid

Material tested	Type of Test	Type of Specimens + Dimension (mm)	Initial Curing		No. of Specimens tested for each storage condition			Soaking period after drying (days)
			Moist Curing (days)	Drying at 105°C (days)	Sealed as a reference (SE)	Soaked in water (SW)	Soaked in crude oil (SO)	
HCP AND MORTAR	Compressive Test	Cubes (50) and Prisms (50x50x150)	7	3	4	4	4	1, 140, 300
	Direct Tensile Test	Briquet ASTM (C92-176)	7	3	6	6	6	150
	Modulus of Elasticity	Prisms (50x50x150)	7	3	3	3	3	300
CONCRETE	Compressive Test	Cubes (100) Cylinders (100x200) Prisms (100x100x500)	7	14	3	3	3	1,70,160,300 and 600
	Flexural Test	Prisms (100x100x500)	7	14	3	3	3	1,70,150,225 and 600
	Splitting Test	Cylinders (100x200)	7	14	3	3	3	1, 60, 300, 470
	Direct Tensile Test	Prisms (50x50x400)	7	14	5	5	5	50, 200, 300 and 455
	Volume Change	Prisms (100x100x300)	7	14	3	3	3	1,3,7,14,25, 45,100,145,200, 366,550
	Pulse Velocity	Cubes (100)	7	14	3	3	3	600
	Modulus of Elasticity	Prisms (100x100x500)	7	14	3	3	3	1,70,160,300 and 600

Table (4.1) Details of the specimens to be tested throughout the investigation for each mix design.

eccentricity, as shown in plate (3.2), the load being applied at a rate of 1.5 $N/mm^2/min$.

The elastic modulus samples were 50x50x150 mm prisms. Two demec discs, 50 mm apart, were fixed at the centre of two opposite sides on each sample and strain readings were taken using a demec gauge sensitive to 1.98×10^{-5} mm/mm. To obtain a flat surface and produce a uniform contact between the platen of the testing machine and the upper surface of the prism specimens, a thin layer of plastic padding was placed on the specimen's top surface and covered with a sheet of thin aluminium foil. The plastic padding was then allowed to harden under a load of 1KN (0.4 N/mm^2) in the testing machine. A stress-strain curve was drawn, and a tangent modulus was obtained at 10 N/mm^2.

4.2.2 Plain Concrete.

4.2.2.1 The compressive strength.

The compressive strength was determined from cube tests according to B.S.1881:1970(103). Cylinders of $\frac{L}{d} = 2$ and prisms of $\frac{L}{w} = 3$ were also used to minimize the effect of the testing machine(46,47,48) and to clarify the real effect of absorbed oil on the true uniaxial compressive strength. The top surface of the cylinder specimens was capped using the same procedure described for the hardened cement paste and mortar specimens. The specimens were tested at a loading rate of 15 N/mm^2 per min.

4.2.2.2 The tensile strength.

The flexural and splitting tensile specimens were made and tested as specified by B.S.1881:1970(103). The flexural beams were tested at the appropriate age by third point loading on an effective span of 406.4 mm (16 in), and loading rate of 3.5 N/mm^2 per min. The splitting-tensile cylinders were tested at a uniform rate of loading, 4.7 N/mm^2 per min, and 300 x 100 mm wide hardboard packing strips were placed between the concrete cylinder and the platens of the testing machine to ensure uniform distribution of load. Particular attention was given to the alignment of the packing strips and the centering of the specimen in the testing machine.

The direct tensile tests were made on concrete prisms as described in section (3.5.3) and fig. (3.4). The experimental difficulties of this test, such as the loading position relative to the axis, led to variations in the test results and are described by Elvery et al.$^{(128)}$. The early tests produced fracture near one end, but this was overcome by enlarging the ends of the specimens. The end caps were fixed using Swiftcrete cement mortar. Considerable care was taken to avoid eccentricity during the fixing of the end caps to the specimen by using a steel frame. The specimens were loaded through flexible cables as shown in plate (3.3).

4.2.1.3 The static modulus of elasticity.

The elastic modulus was obtained from uniaxial compression tests carried out according to B.S.1881:1970 on three prisms (table 4.1) at each age. Strains were measured using a 100 mm demec gauge at the centre of two opposite sides of each prism. The gauge sensitivity was 1.65x10^{-5} mm/mm. A tangent elastic modulus was determined from the stress-strain curve at 10 N/mm^2.

Poisson's ratio was also determined from the same test using two additional demec discs at 50 mm apart fixed in the lateral direction at the centre of the specimen.

4.2.2.4 Ultrasonic-pulse velocity test.

In this test the pulse velocity technique was used to detect the onset of cracking in a cube compressive test. The apparatus consists of the "Pundit" transit time display counter, two transducers (50 mm in diameter) which have a frequency of 50 kHz, and two transducer leads. Pulses emitted by one transducer, placed in contact with one side of the specimen, are transmitted through the concrete and received by a second transducer on the opposite side. An electronic timing device in the apparatus measures the transit time to within about 0.15 μs. By measuring the time and the width of the specimen (129), it was possible to calculate the velocity of the ultrasonic pulse through the concrete. When a crack of sufficient size has formed, the pulse has to circumvent the crack and hence requires longer time to traverse the specimen. This caused an apparent decrease in the pulse velocity.

After positioning the specimen and zeroing the electronic equipment, the transducers were placed on the centre of either side of the specimen. The pulse transit time was taken while the specimen was being loaded to failure in increments of 10 KN. A load-velocity curve was drawn and the load at which the crack extension began was determined.

4.2.2.5 Volume change measurements.

In the present investigation, the behaviour of dry concrete prisms soaked in crude oil was compared with the behaviour of similar companion specimens dried and soaked in water, and others dried and sealed in polythene bags in the open air in the same laboratory. All the concrete specimens were cast in 100x100x300 mm steel moulds, and cured as described in the previous chapter (Section 3.6).

Shrinkage strains were measured using a measuring apparatus incorporating a micrometer gauge accurate to 0.0025 mm as described in B.S.1881: 1970. For each specimen, a shallow depression 3 mm deep was drilled into the centre of each end of the prism, and a stainless steel ball cemented onto each end of the axis of the prism with plastic padding. During water curing, the stainless steel balls were covered with a petroleum jelly as an extra protection against rusting of the steel.

4.3 TEST RESULTS FOR HARDENED CEMENT PASTES AND MORTARS.

4.3.1 Introduction.

Because many factors influence the strength of concrete and the variation that is obtained, individual results are not a reliable basis for action and it is important that a statistical consideration of the accuracy of the observed results is known.

The maximum error, for a 95% probability level, by which an average of a given number of tests differs from the true unbiased average, is calculated using the following formula(130) :-

$$E_r = \frac{V}{\sqrt{n}} t \qquad \cdots\cdots \qquad (4.1)$$

where E_r is the maximum error of the average of the sample (per cent),(t) is the student's (t) for (n-1) degree of freedom at a specific probability level, (V) is the coefficient of variation and n, the number of tests.

The student's t test is applied in this investigation to determine whether the observed difference between the true averages of measured properties of oil saturated and reference specimens is caused by the random variation of the individual test results or is attributed to some inherent difference between the two concretes. The 5% level of significance is used to test the accuracy of the test results using the values of t given by Davis and Goldsmith(131). If the probability (P) is equal to or less than 5%, this indicates strong evidence of a significant difference at that level.

The test results presented in sections (4.4,4.5) will contain the 95% confidence limit of each sample mean and the significance of the difference of any two required means at the 0.05 level.

4.3.2 Effect of Oil Saturation on the Cube Compressive Strength of HCP's and Mortars.

4.3.2.1 Effect of Oil Saturation on the Cube Compressive Strength at Various Ages.

The test specimens were dried at 105° C to a constant weight so that all evaporable water was removed and hydration of the paste stopped(132). Any change in the strength was therefore due to the crude oil absorbed by the specimens. The compressive strength of the dry cubes soaked in oil for different periods was compared to those soaked in water and those sealed as a reference. The test results are given in table (4.2) and plotted in figs. (4.1,4.2,4.3). The following observations are made:-

Material	w/c ratio	7-days Moist-Curing	1-day			140-days			630-days		
			Sealed (SE)	Soaked in oil (SO)	Soaked in water (SW)	Sealed (SE)	Soaked in oil (SO)	Soaked in water (SW)	Sealed (SE)	Soaked in oil (SO)	Soaked in water (SW)
HCPs	0.30	94.0 (±5.8)*	106.0 (±7.8)	104.0 (±9.3)	60.0 (±3.6)	110.0 (±7.5)	102.0 (±10)	97.5 (±4.5)	113.5 (±6.5)	101.5 (±7.5)	127.5 (±4)
	0.35	78.0 (±4.7)	92.0 (±6.5)	90.0 (±6.1)	46.5 (±4.3)	94.0 (±8.7)	86.5 (±7.7)	88.0 (±6.8)	98.0 (±5.4)	85.0 (±7.1)	112.5 (±6.1)
	0.40	58.0 (±5.6)	69.0 (±5.1)	66.0 (±5.1)	28.5 (±6.9)	70.5 (±6.3)	63.5 (±5.8)	72.5 (±6.7)	75.5 (±5.7)	63.0 (±7.5)	88.0 (±7.3)
1:2 Mortar	0.40	55.5 (±2.9)	65.9 (±2.1)	68.0 (±3.4)	42.5 (±4.3)	71.5 (±6.5)	67.5 (±7.8)	69.7 (±7.3)	73.5 (±4.6)	65.0 (±6.9)	80.0 (±7.4)
	0.50	49.5 (±2.7)	60.4 (±2.5)	60.7 (±4.4)	36.5 (±3.9)	63.9 (±5.8)	59.3 (±6.3)	60.3 (±7.9)	65.5 (±3.9)	58.5 (±7.2)	71.5 (±7.9)
	0.60	39.5 (±4.1)	48.5 (±3.1)	49.5 (±3.4)	27.5 (±5.4)	50.8 (±4.0)	48.5 (±7.1)	52.5 (±4.7)	54.0 (±4.7)	46.0 (±7.6)	60.0 (±8.1)
1:3 Mortar	0.50	43.5 (±2.9)	51.5 (±4.0)	53.5 (±4.9)	33.5 (±4.2)	53.0 (±2.8)	51.5 (±5.8)	49.5 (±5.0)	56.0 (±5.8)	49.5 (±4.5)	62.5 (±3.7)
	0.60	36.5 (±3.2)	45.5 (±3.9)	46.5 (±2.9)	27.0 (±5.1)	47.0 (±2.7)	43.5 (±6.1)	41.5 (±4.0)	49.3 (±5.1)	41.7 (±5.2)	54.9 (±3.8)
	0.70	30.0 (±4.7)	38.0 (±3.9)	39.5 (±4.3)	20.5 (±3.9)	39.5 (±5.4)	37.5 (±7.6)	35.5 (±3.9)	41.2 (±3.8)	34.5 (±3.3)	48.0 (±3.1)

*95% Confidence limits are given in brackets.

Table (4.2) Compressive Strength of HCPs and Mortar at Various ages, w/c ratios and Storage Conditions - N/mm^2.

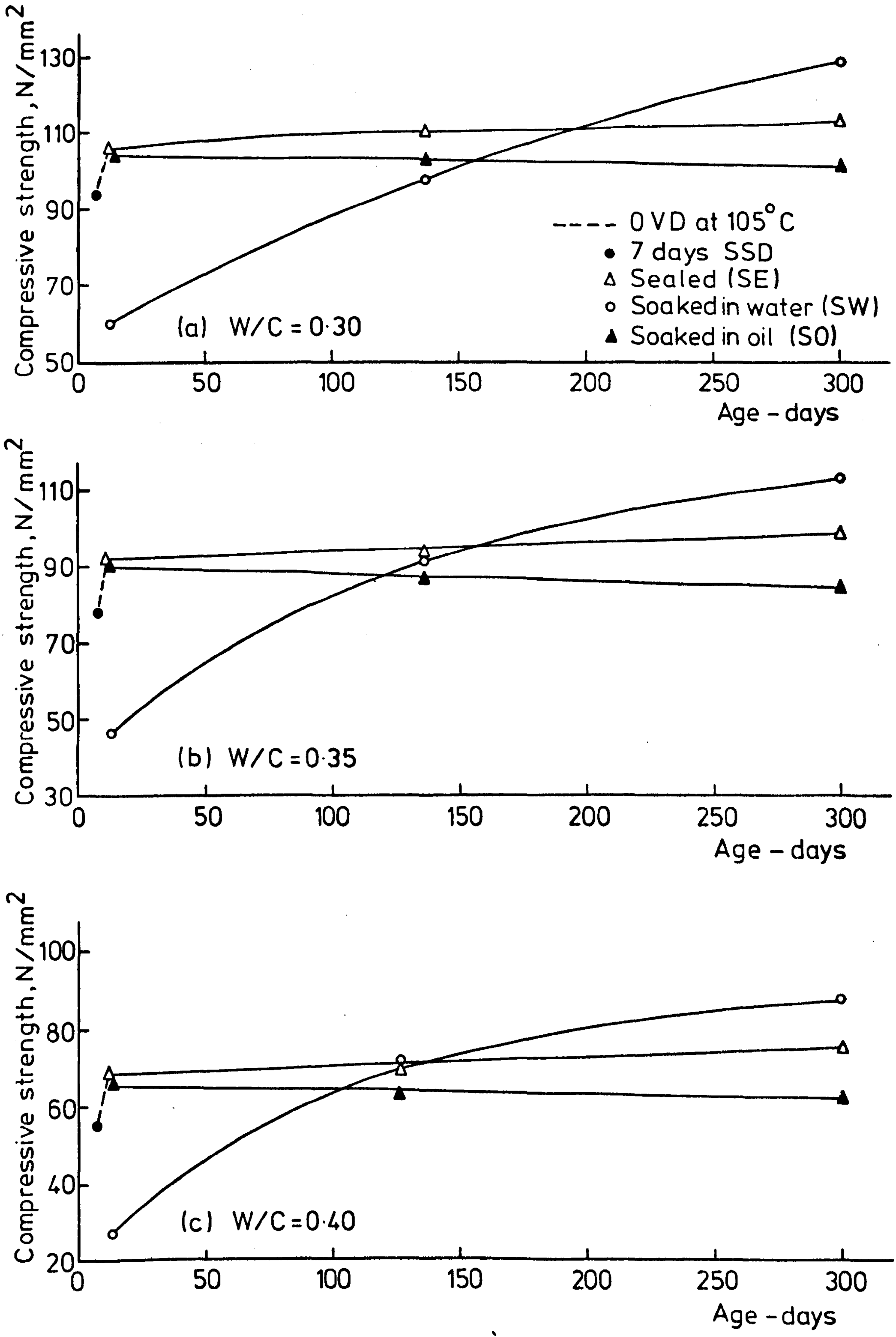


FIG. 4-1 EFFECT OF STORAGE CONDITIONS ON THE COMPRESSIVE STRENGTH OF PASTES AT VARIOUS AGES AND W/C

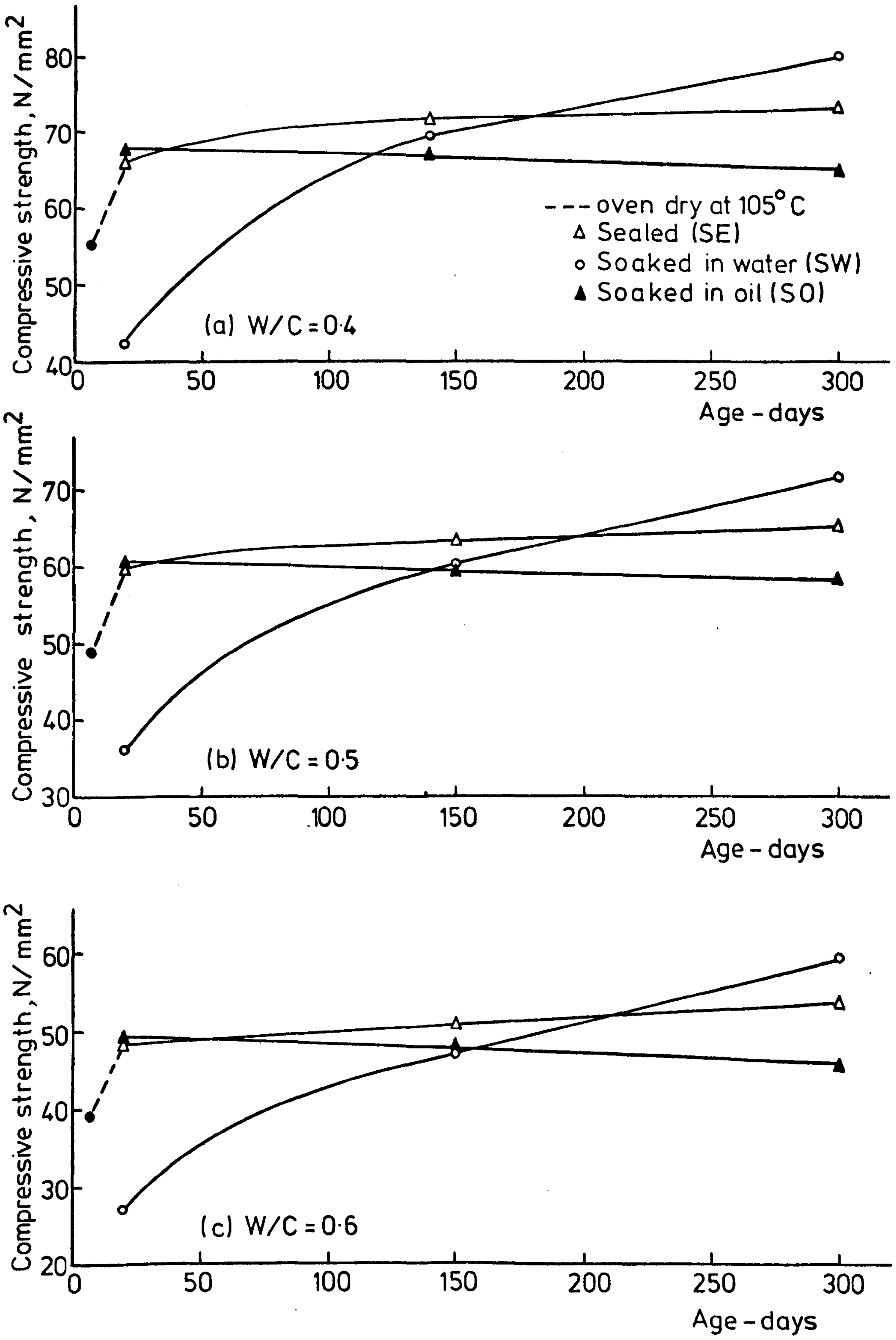
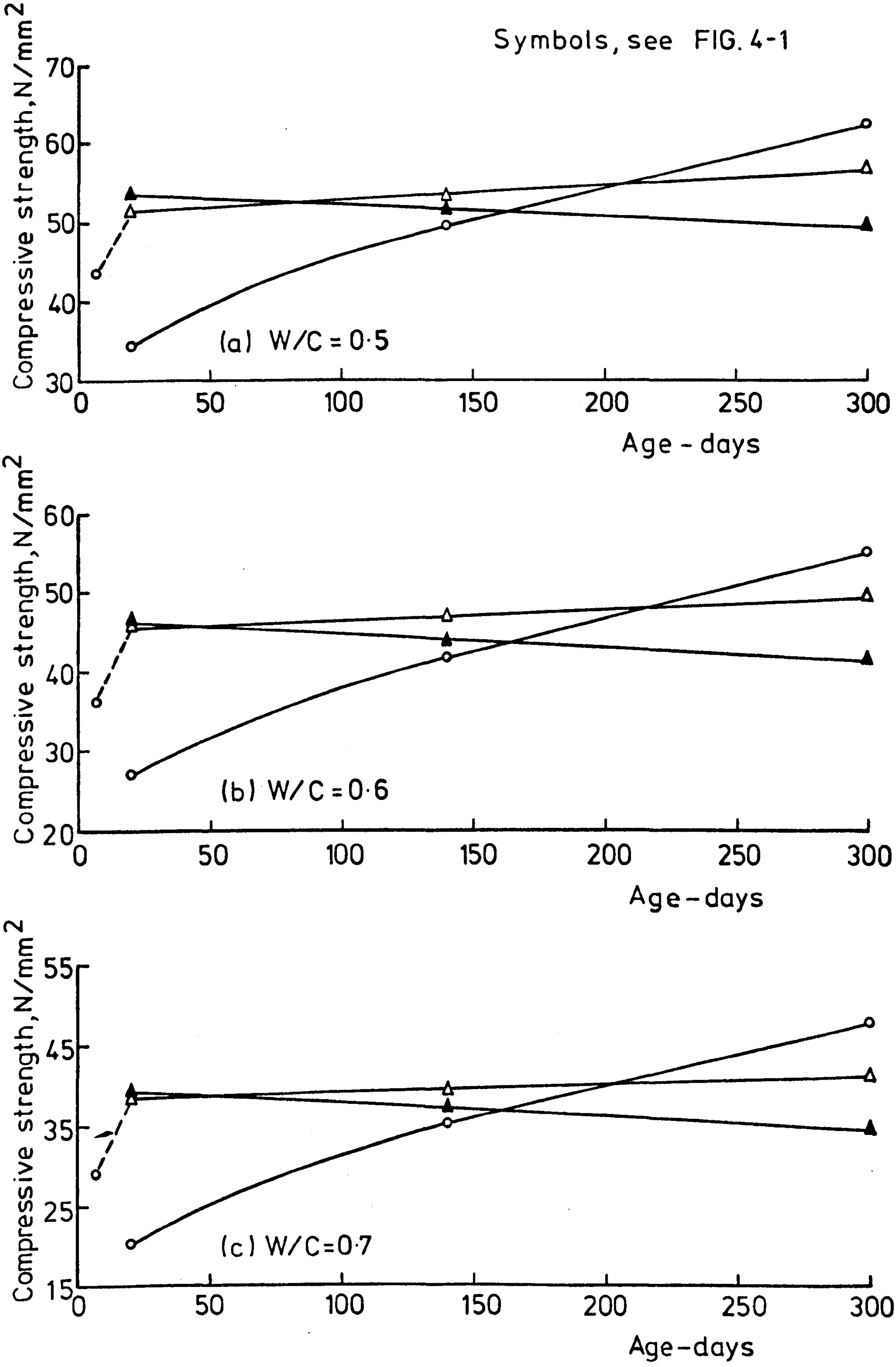


FIG. 4-2 EFFECT OF STORAGE CONDITIONS ON THE COMPRESSIVE STRENGTH OF 1:2 MORTARS AT VARIOUS AGES AND W/C



Symbols, see FIG. 4-1

FIG. 4-3 EFFECT OF STORAGE CONDITIONS ON THE COMPRESSIVE STRENGTH OF 1:3 MORTARS AT VARIOUS AGES AND W/C RATIO

1. There was a significant ($P \leqslant 0.05$) increase in the strength of the HCP and mortar after drying in the oven at 105°C. This increase varied between 12.5-19% for HCP's and 18.5-26.5% for mortars. The largest increase was for the mixes of higher w/c ratio.

2. The compressive strength of the sealed specimens increases with time. This significant ($P < 0.05$) increase varied between 7-9% for cement pastes and 7-11% for mortars within a storage period of 300 days. This increase in strength was accompanied by an increase in the weight of the specimens due to absorption of water from the surrounding environment.

3. The strength of dry cube specimens was reduced by between 40-60% for HCPs and 35-45% for mortars after 24 hours of soaking in water. This reduction was then followed by an initially high rate of strength gain which gradually decreased with time. This increase could be due to re-hydration after being resoaked in water.

4. It was observed that the strength of oil saturated cubes was decreasing in direct relation to the amount of oil absorbed. The maximum reduction was significant at the 0.05 level and varied between 4-8.5% for HCPs (with an oil content variation of 8-15% by weight), 4.5-10% for 1:2 Mortars (with an oil content variation of 5.5-9% by weight), and 6-14% for 1:3 Mortars (with an oil content variation of 5-9% by weight) within a soaking period of 300 days, as a percentage of 1-day reference strength.

4.3.2.2 Effect of Oil Saturation on the Cube Compressive Strength at Various W/C Ratios.

The variation of cube strength with w/c ratio is illustrated in fig. (4.4), both for HCPs and for Mortars in relation to different storage conditions. These results suggest the following observations:

1. The compressive strength of crude oil saturated cubes was lower for all w/c ratios than both water saturated and reference samples. The percentage of reduction increased as the w/c ratio increased. The reduction in the strength over that of the reference samples was between 13-20% for HCPs and 11.5-16 for Mortars.

2. The relationship between compressive strength and w/c ratio for the test results presented in fig. (4.4) can be expressed exponentially in the form:

$$fc = a\,e^{b(w/c)} \qquad \ldots\ldots \qquad (4.2)$$

The values of a and b were obtained for the dry samples resaturated in water and those resaturated in oil are given in table (4.3).

Material	Curing Condition	a	b	r*
HCP	300 days - SO	434	-4.77	98.64
	" - SW	395	-3.71	98.63
1:2 Mortar	300 days - SO	133	-1.73	97.90
	" - SW	144	-1.44	99.74
1:3 Mortar	300 days - SO	122	-1.81	99.82
	" - SW	121	-1.32	99.95

r* = correlation coefficientx100%

Table (4.3) Constants of Regression - eqn. (4.2) compressive strength Vs w/c Ratio for HCP & Mortar.

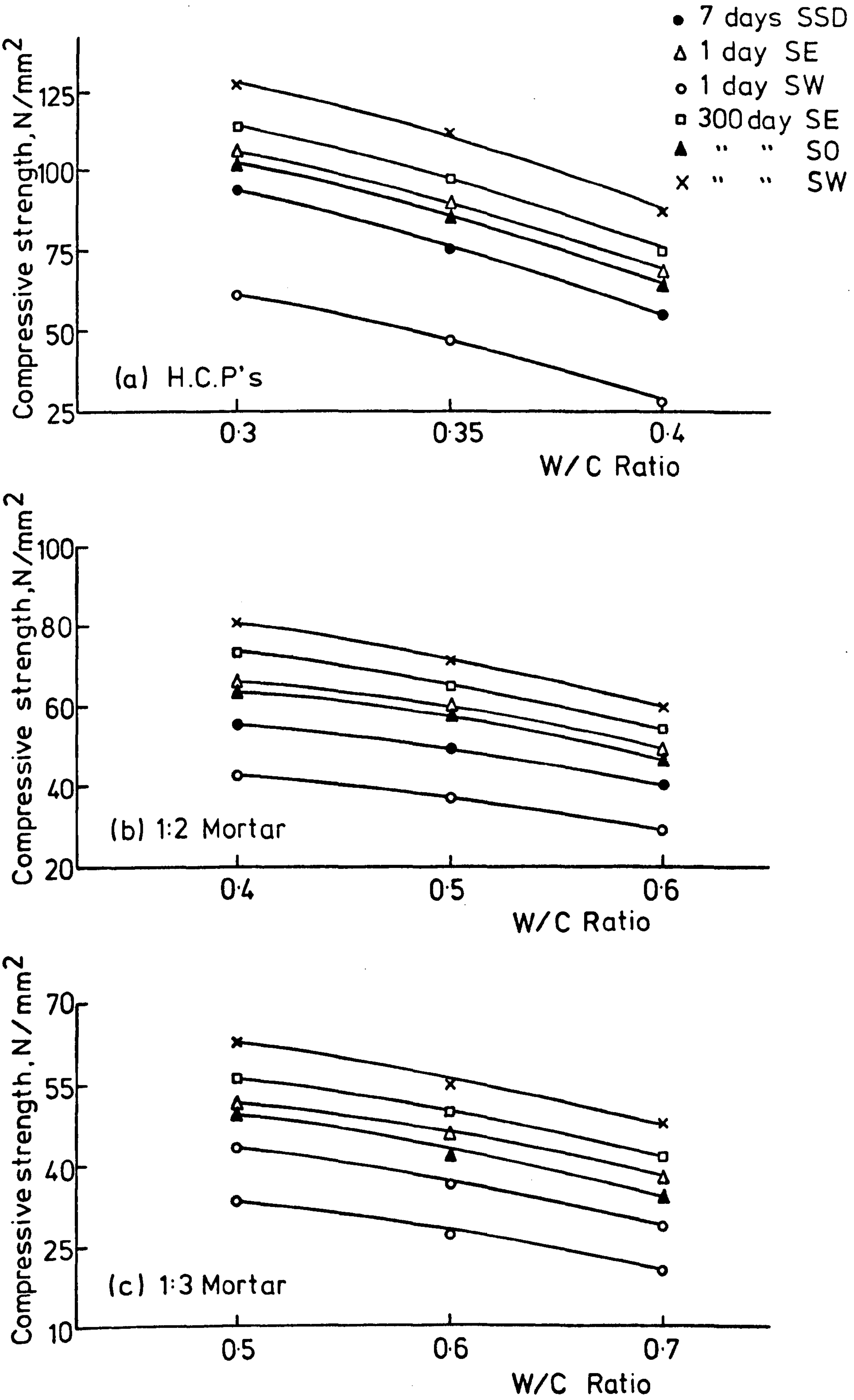


FIG. 4-4 VARIATION OF COMPRESSIVE STRENGTH WITH W/C RATIO AND STORAGE CONDITIONS

4.3.3 Static Modulus of Elasticity of HCPs and Mortars.

4.3.3.1 Stress-Strain Curves.

Typical stress-strain curves are shown in fig.(4.5) for HCPs, and in fig. (4.6,4.7) for 1:2 Mortar and 1:3 Mortar respectively. From these curves the following is observed:

1. The curves were generally non-linear and as expected the non-linearity increased with an increase in w/c ratio. The non-linearity was also affected by the curing process of the samples. Dry specimens resoaked in water for 300 days show higher strength and exhibited greater linearity and stiffness than those saturated in oil or sealed as a reference.
2. The failure of hardened cement paste specimens was instant and violent and particularly so for pastes of low w/c ratio and those cured in water. The failure of the crude oil saturation specimens was almost similar to the sealed ones.
3. For mortar specimens, the stress-strain diagram was more curved than for the comparable paste of the same w/c ratio. The elastic modulus and failure stress for 1:2 Mortar was higher than that for 1:3 Mortar using the same w/c ratio. The non-linearity of the stress-strain curves slightly increased with prolonged storage in oil compared to that of the sealed specimens tested at the same age.

4.3.3.2 Effect of Crude Oil Saturation on the Elastic Modulus and Prism Strength of HCPs and Mortars.

Table (4.4) gives the modulus of elasticity and failure stress for the HCP and Mortar prisms tested. Three samples were tested to failure at each test and the variation of each individual sample from the mean values was small and of the

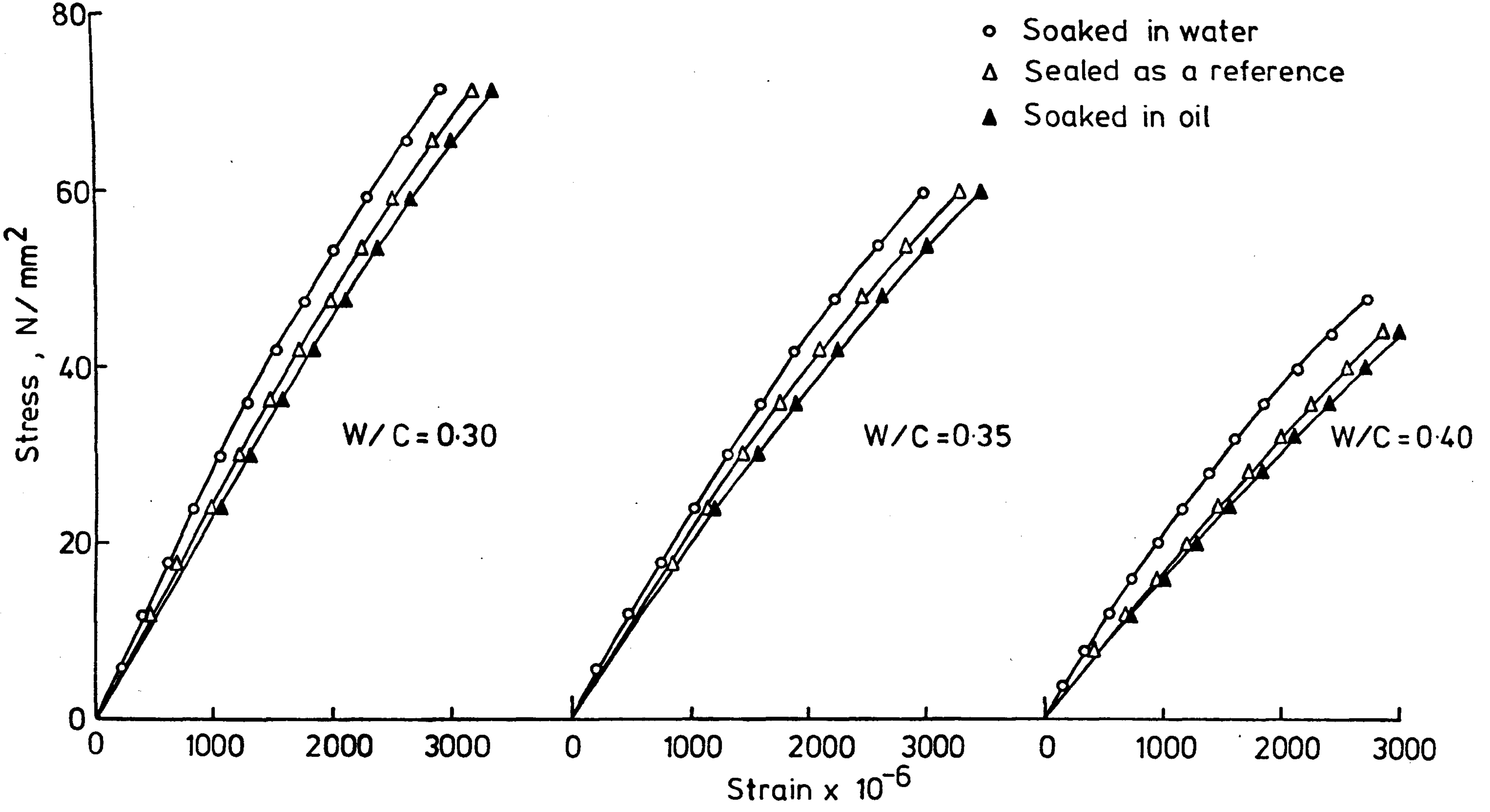


FIG. 4·5 STRESS-STRAIN RELATIONSHIP SHOWING THE EFFECT OF STORAGE CONDITIONS AND W/C RATIO ON HARDEND CEMENT PASTE SPECIMENS (AT 300 DAYS)

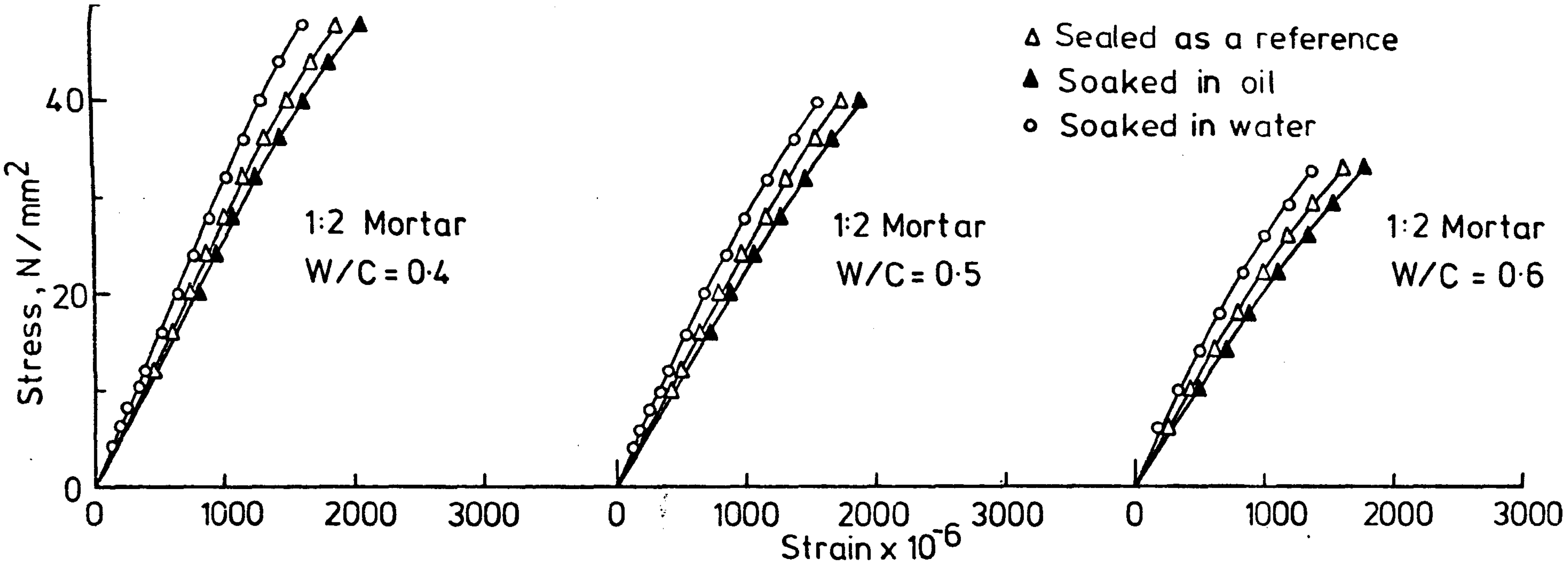


FIG. 4-6 STRESS-STRAIN RELATIONSHIP SHOWING THE EFFECT OF STORAGE CONDITIONS ON MORTAR SPECIMENS (AT 300 DAYS)

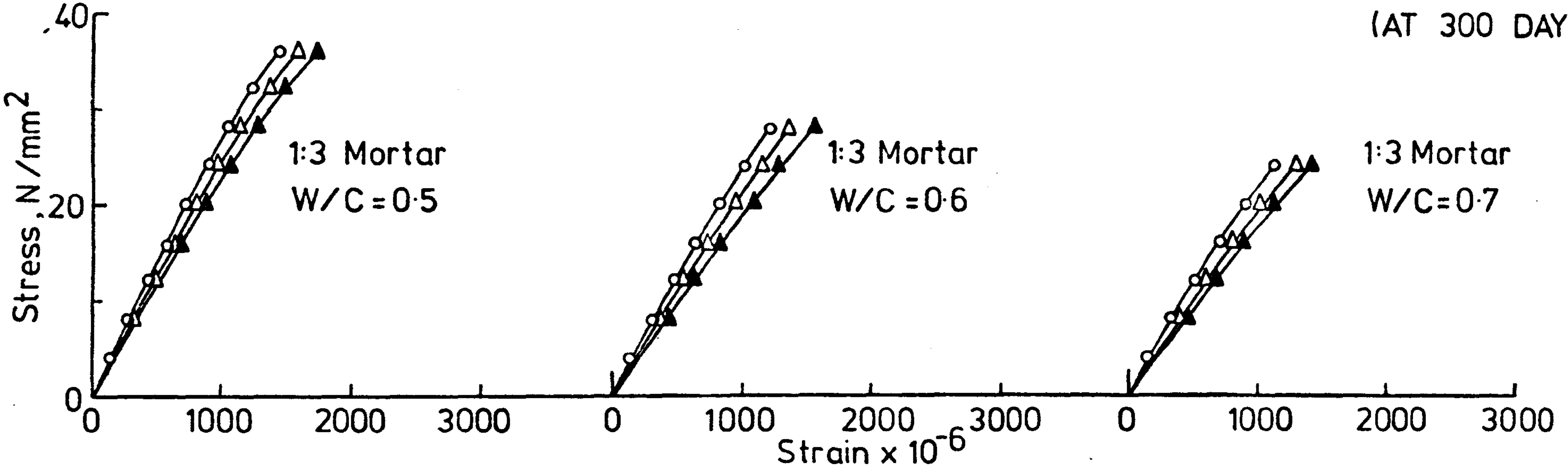


FIG. 4·7 STRESS-STRAIN RELATIONSHIP SHOWING THE EFFECT OF STORAGE CONDITIONS ON MORTAR SPECIMENS (AT 300 DAYS) order of ± 4%. The variation of the elastic modulus with w/c ratio is illustrated in fig. (4.8) and the following is observed:-

1. The test results presented in table (4.4) and fig. (4.8) show that the elastic modulus decreased with an increase in the w/c ratio following an exponential relationship of the form:

$$E = a\,e^{b(w/c)} \quad ..... \quad (4.3)$$

The values of the constants a and b were determined by regression analysis and are given in table (4.5).

2. The storage condition had a considerable effect on the values of the elastic modulus. The oil saturated specimens have the lowest elastic modulus, e.g., the dried HCP specimens resaturated in water for 300 days and the companion sealed specimens were higher than the modulus of oil saturated specimens by about 20-26% and 7-10% respectively.

3. The variation of failure stress with w/c ratio was similar in form to those obtained for elastic modulus, e.g., HCP specimens resoaked in water for 300 days were between 16-18% higher than the sealed specimens tested at the same age while oil saturated specimens were between 5-7% less. Reductions in Mortar specimens in prolonged storage in oil were between 8.5-12% for 1:2 Mortars and 10-15% for 1:3 Mortars less than that of the corresponding sealed specimens.

4.3.4 Effect of Oil Saturation on the Direct Tensile Strength of HCPs and Mortars.

The direct tensile strengths are given in table (4.6) and plotted in fig.(4.9) and were the average of at least 6 specimens.

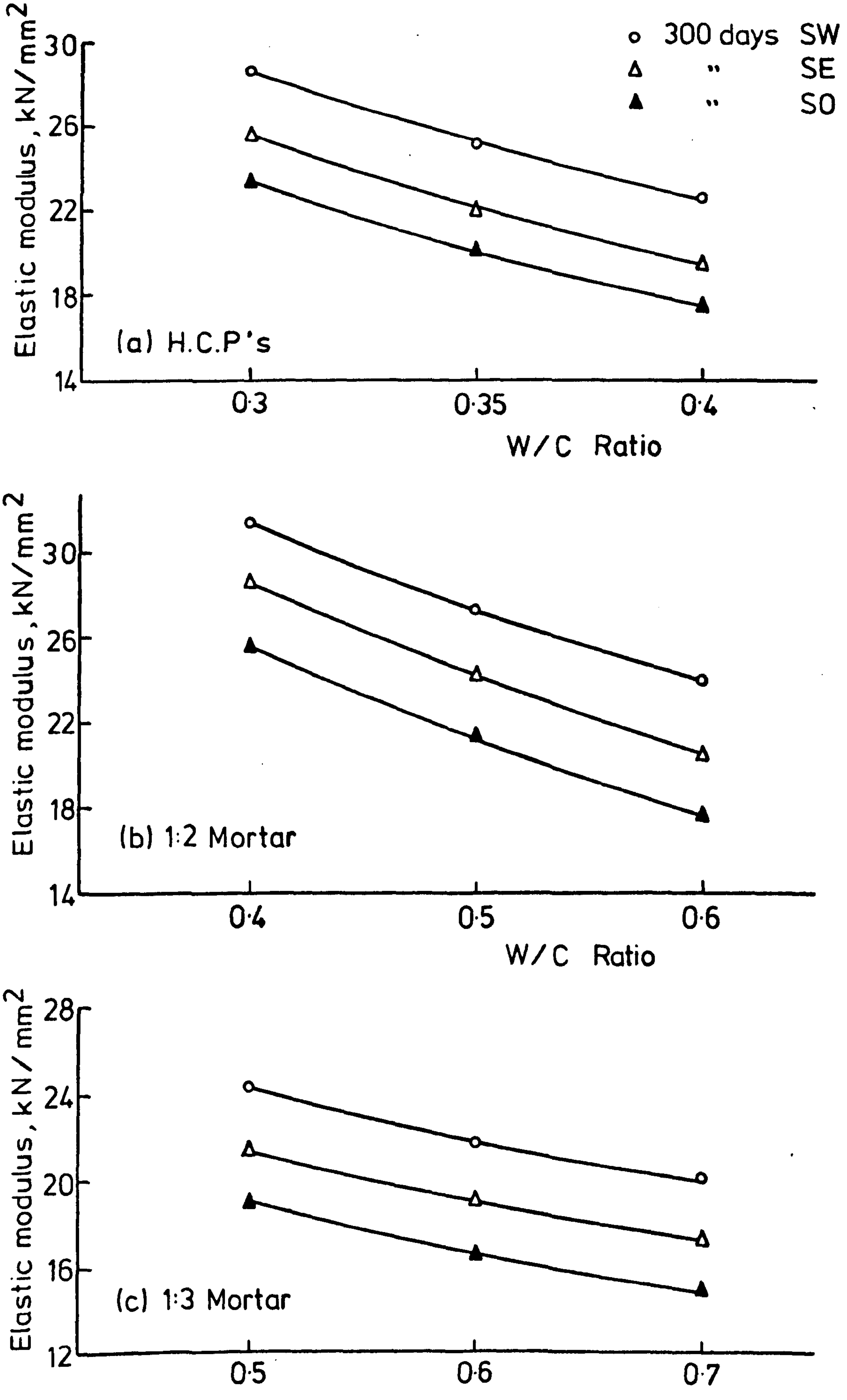


FIG. 4·8 EFFECT OF STORAGE CONDITIONS AND W/C RATIO ON THE ELASTIC MODULUS OF H.C.P's AND MORTARS

Materials	w/c Ratio	Sealed (SE)		Soaked in oil(RSO)		Soaked in water (RSW)	
		E KN/mm^2	fc N/mm^2	E KN/mm^2	fc N/mm^2	E KN/mm^2	fc N/mm^2
HCP	0.30	25.6	98.0	23.5	92.5	28.5	114.0
	0.35	22.0	84.5	20.5	79.2	25.0	99.5
	0.40	19.5	71.8	17.8	66.9	22.5	84.5
1:2 Mortar	0.40	28.5	69.0	25.6	64.5	31.3	77.0
	0.50	24.3	60.0	21.3	53.4	27.2	66.8
	0.60	20.5	49.5	17.6	43.5	24.0	56.6
1:3 Mortar	0.50	21.5	55.5	19.0	49.8	24.4	60.1
	0.60	19.0	47.6	16.6	40.8	21.7	52.2
	0.70	17.3	40.8	15.1	34.7	20.1	45.3

Table (4.4) Elastic Moduli (E) and Failure stresses (fc) for HCPs and Mortars.

Material	Storing condition	a	b	r
HCP's	Resoaked in water.	57.6	- 2.36	99.31
	Sealed as reference.	57.6	- 2.72	99.86
	Resoaked in oil.	57.1	- 2.95	99.91
1:2 Mortar	Resoaked in water.	53.1	- 1.328	99.90
	Sealed as reference.	55.1	- 1.647	99.91
	Resoaked in oil.	54.2	- 1.873	99.91
1:3 Mortar	Resoaked in water.	39.0	- 0.969	99.28
	Sealed as reference.	36.9	- 1.087	99.83
	Resoaked in oil.	34.0	- 1.175	98.95

Table (4.5) Constants of regression - eqn. (4.3) Elastic modulus Vs. w/c ratio.

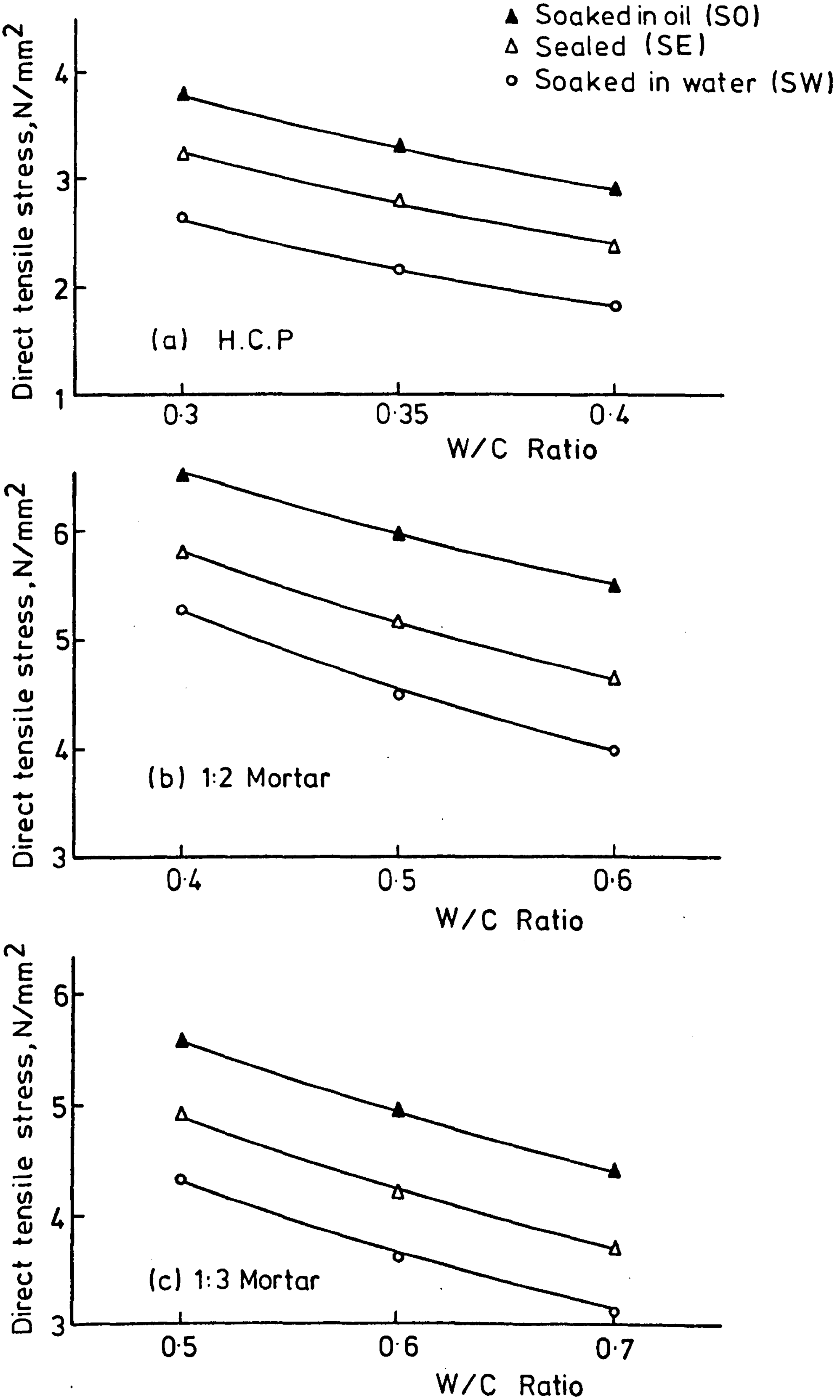


FIG.4-9 THE EFFECT OF STORAGE CONDITIONS ON THE DIRECT TENSILE STRENGTH OF H.C.P's AND MORTARS OF VARIOUS W/C (AT 150 DAYS SOAKING PERIOD)

There was a big scatter in the test results which varies between 6-12.5% with an average value of 8.5%. From the test results it is observed that:

1. The direct tensile strength of the dried specimens resoaked in water was lower than the oil saturated samples or those sealed as a reference. The reduction in the value over that of the reference samples was between 18-22% for HCPs, 9-14% for 1:2 Mortars, and 12-16 for 1:3 Mortars after a soaking period of 150 days.

2. The companion oil soaked specimens showed a significant increase ($P \leqslant 0.5$) in the direct tensile strength compared to those of the reference specimens. The variation in this increase was between 16-22% for HPCs (with an oil content variation of 8-14% by weight), 12-16% for 1:2 Mortars (with an oil content variation of 4.5-8% by weight), and was 14-20% for 1:3 Mortars (with an oil content variation of 6-9% by weight). The high percentage of increase was for the specimens of higher w/c ratio which had the highest porosity and thus contained the largest amount of absorbed oil.

Materials	w/c ratio	Sealed (SE)		Soaked in oil (SO)		Soaked in water (SW)	
		ft.N/mm^2	S.D.	ft.N/mm^2	S.D.	ft.N/mm^2	S.D.
HCP	0.30	3.23(±.25)	0.24	3.75(±.32)	0.31	2.63(±.19)	0.18
	0.35	2.78(±0.30)	0.29	3.33(±.43)	0.41	2.20(±.25)	0.24
	0.40	2.36(±.18)	0.17	2.90(±.33)	0.32	1.83(±.17)	0.16
1:2 Mortar	0.40	5.80(±.33)	0.31	6.53(±.41)	0.39	5.28(±.39)	0.37
	0.50	5.15(±.29)	0.28	5.93(±.50)	0.49	4.50(±.41)	0.39
	0.60	4.63(±.26)	0.25	5.42(±.44)	0.42	3.98(±.41)	0.39
1:3 Mortar	0.50	4.93(±.26)	0.25	5.61(±.38)	0.36	4.33(±.29)	0.28
	0.60	4.20(±.29)	0.26	4.95(±.27)	0.28	3.63(±.23)	0.22
	0.70	3.70(±.24)	0.23	4.43(±.30)	0.29	3.13(±.20)	0.19

Table (4.6) Direct Tensile Strength (ft) and Standard Deviation (S.D.) for HCPs and Mortars.

4.4 TEST RESULTS FOR PLAIN CONCRETE.

4.4.1 Effect of Oil Saturation on the Compressive Strength of Concrete.

4.4.1.1 Effect of Prolonged Storage in Crude Oil on the Cube Crushing Strength of Concrete.

The specimens used here had mix proportions as shown in table (3.5) and were cured as described in section (3.6). The test results for the different types of curing condition were the mean of three cube results. Variation of individual sample strength from the mean value was small with a maximum difference of about ± 5% and the average difference about ± 2.6%. The test results are given in tables (4.7, 4.8) and plotted as a function of age as shown in figs. (4.10, 4.11, 4.12). The following observations are made:-

1. The specimens which were kept continuously cured in water after demoulding and tested in a saturated surface dry condition, showed a continuous increase in strength with age. After 600 days of moist curing, the increase in the compressive strength was between 16-18% higher than that at 28 days moist curing.

2. Oven drying at 105° C increased the strength between 26-31% for Mix A and 25-30% for Mix B over the 7-day moist curing strength. The cube crushing strength of the oven dried specimens sealed for 600 days had a significant ($P \leqslant 0.05$) reduction in strength between 8-16.5%. The reduction was higher for Mix B and for higher w/c ratio.

3. The strength of oven dry concrete specimens was reduced by between 25-36% after 24 hours of soaking in water. The

Type of Specimen Tested	Mix Designation	w/c Ratio	7-days		28-days		150-days		240-days		400-days	
			Strength N/mm^2	Coeff.of Variation	Strength N/mm^2	Coeff.of Variation	Strength N/mm^2	Coeff.of Variation	Strength N/mm^2	Coeff.of Variation	Strength N/mm^2	Coeff. of Variation
CUBES (100 mm)	A1	0.4	62.5(±3.2)*	2.04	74.0(±4.6)	2.54	81.0(±4.9)	2.42	83.0(±3.6)	1.73	86.5(±4.0)	1.87
	A2	0.5	52.0(±1.6)	1.19	61.0(±2.3)	1.51	65.5(±4.2)	2.56	68.0(±2.0)	1.19	71.5(±5.0)	2.81
	A3	0.6	41.5(±3.6)	3.50	50.0(±2.9)	2.37	55.1(±4.4)	3.20	57.5(±3.3)	2.31	59.0(±2.2)	1.47
	B1	0.4	62.0(±3.4)	2.23	73.5(±2.5)	1.39	80.0(±3.7)	1.84	83.0(±5.0)	2.40	85.6(±2.5)	1.16
	B2	0.5	51.0(±1.8)	1.39	60.2(±3.4)	2.31	65.0(±5.3)	3.27	67.5(±2.8)	1.68	69.5(±5.4)	3.15
	B3	0.6	40.7(±2.3)	2.24	49.0(±4.4)	3.63	53.5(±4.3)	3.26	55.5(±2.6)	1.90	57.0(±4.2)	2.99
CYLINDERS (100 x 200 mm)	A1	0.4	52.5(±4.4)	3.34	60.0(±4.0)	2.66	67.3(±4.7)	2.79	68.5(±4.9)	2.86	72.0(±6.1)	3.42
	A2	0.5	39.5(±2.1)	2.15	49.5(±2.0)	1.66	55.0(±3.0)	2.16	57.0(±3.4)	2.41	60.0(±5.7)	3.81
	A3	0.6	29.3(±1.8)	2.50	39.0(±2.8)	2.93	43.0(±2.4)	2.20	46.0(±1.9)	1.72	48.5(±4.2)	3.47
	B1	0.4	49.8(±3.6)	2.90	57.0(±5.1)	3.62	66.5(±3.7)	2.22	68.5(±2.2)	1.28	70.5(±4.5)	2.55
	B2	0.5	37.5(±3.1)	3.28	47.5(±2.8)	2.35	54.5(±2.7)	2.01	56.5(±3.0)	2.12	58.8(±2.8)	1.91
	B3	0.6	30.0(±2.5)	3.39	38.5(±1.8)	1.93	44.0(±6.5)	5.91	46.5(±3.6)	3.14	48.0(±6.0)	4.14
PRISMS (100x100 x300 mm)	A1	0.4	50.0(±3.2)	2.55	58.0(±3.8)	2.62	67.0(±4.0)	2.40	69.5(±2.2)	1.30	72.5(±3.8)	2.08
	A2	0.5	40.0(±1.8)	1.83	47.5(±3.9)	3.27	54.0(±1.7)	1.27	57.0(±5.5)	3.80	60.5(±4.0)	2.72
	A3	0.6	30.5(±2.4)	3.20	37.0(±3.3)	3.54	43.0(±4.2)	3.92	45.0(±5.4)	4.79	58.0(±3.7)	3.13
	B1	0.4	49.0(±3.0)	2.50	55.5(±4.2)	2.99	63.0(±3.1)	1.99	65.0(±3.3)	2.04	69.5(±2.7)	1.54
	B2	0.5	36.5(±2.6)	2.85	45.3(±4.3)	3.85	53.5(±3.4)	2.54	56.2(±3.7)	2.64	5.80(±3.7)	2.54
	B3	0.6	29.5(±2.5)	3.45	36.5(±4.3)	4.96	42.5(±4.6)	4.38)	46.0(±5.4)	4.76	47.5(±5.9)	5.06

*95% Confidence limits are given in brackets.

Table (4.7) <u>Compressive Strength of Concrete Cubes, Cylinders, and Prisms continuously Cured in Water for Various Ages - N/mm^2.</u>

Type of Specimens Used	Mix Designation	w/c Ratio	1-day			70-day			160-day			300-day			600-day		
			Sealed (SE)	Soaked in oil (SO)	Soaked in water (SW)	Sealed (SE)	Soaked in oil (SO)	Soaked in water (SW)	Sealed (SE)	Soaked in oil (SO)	Soaked in water (SW)	Sealed (SE)	Soaked in oil (SO)	Soaked in water (SW)	Sealed (SE)	Soaked in oil (SO)	Soaked in water (SW)
CUBES (100 mm)	A1	0.4	79.0 (±2.4)*	78.0 (±2.9)	59.0 (±5.2)	77.6 (±2.1)	77.0 (±4.1)	68.5 (±4.2)	76.0 (±2.3)	74.5 (±5.3)	74.0 (±2.8)	74.0 (±2.3)	72.5 (±5.6)	78.5 (±2.8)	72.5 (±3.4)	71.5 (±4.6)	82.5 (±5.3)
	A2	0.5	67.0 (±2.9)	66.0 (±3.3)	44.0 (±6.3)	64.5 (±2.3)	63.5 (±4.7)	51.5 (±6.6)	60.8 (±5.7)	59.5 (±2.8)	57.5 (±5.8)	58.5 (±2.1)	57.5 (±2.2)	61.5 (±5.3)	57.8 (±3.1)	56.5 (±4.5)	67.2 (±6.4)
	A3	0.6	54.0 (±5.7)	52.5 (±4.3)	34.5 (±6.4)	51.0 (±4.2)	50.0 (±4.5)	42.6 (±4.5)	48.5 (±3.8)	46.0 (±6.1)	46.0 (±5.8)	46.5 (±4.7)	44.0 (±7.3)	49.0 (±4.5)	45.0 (±2.5)	43.0 (±7.4)	55.8 (±8.2)
	B1	0.4	77.5 (±3.8)	76.3 (±6.3)	57.0 (±5.6)	76.0 (±2.7)	75.4 (±6.9)	67.5 (±3.3)	75.0 (±4.8)	73.0 (±6.0)	74.0 (±5.3)	73.0 (±5.0)	71.0 (±3.8)	76.5 (±5.9)	71.5 (±6.9)	69.5 (±6.7)	81.0 (±7.2)
	B2	0.5	65.0 (±4.0)	63.5 (±6.0)	43.0 (±2.4)	62.0 (±6.2)	61.5 (±5.2)	56.5 (±3.6)	59.0 (±5.7)	58.5 (±4.4)	*61.5 (±3.1)	58.0 (±5.2)	56.5 (±6.3)	63.5 (±3.5)	57.0 (±2.1)	54.5 (±5.8)	67.0 (±2.9)
	B3	0.6	53.0 (±6.5)	51.5 (±6.7)	33.5 (±4.4)	50.5 (±6.2)	48.5 (±2.3)	45.5 (±2.1)	47.8 (±3.2)	45.5 (±5.6)	48.5 (±6.8)	44.5 (±6.2)	43.0 (±5.9)	51.5 (±5.3)	44.0 (±3.0)	41.0 (±5.3)	54.6 (±6.4)
CYLINDERS (100 mm dia.x 200 mm)	A1	0.4	65.0 (±2.7)	63.5 (±3.3)	45.9 (±3.5)	63.5 (±3.1)	63.0 (±3.6)	54.0 (±5.1)	60.0 (±4.5)	62.5 (±3.0)	59.5 (±5.0)	59.0 (±5.8)	60.0 (±5.4)	63.5 (±7.1)	57.5 (±4.1)	58.5 (±4.7)	70.0 (±2.63)
	A2	0.5	49.0 (±2.6)	47.1 (±3.5)	34.5 (±2.1)	45.0 (±2.0)	46.5 (±4.4)	43.5 (±2.4)	44.2 (±3.4)	45.8 (±3.3)	48.4 (±3.8)	42.7 (±5.6)	44.6 (±4.2)	50.0 (±2.9)	41.5 (±3.5)	42.5 (±3.2)	54.0 (±4.3)
	A3	0.6	38.0 (±0.9)	35.5 (±4.0)	25.0 (±1.7)	35.5 (±6.0)	35.0 (±4.9)	31.5 (±3.1)	32.5 (±2.7)	34.5 (±6.3)	35.0 (±2.7)	23.5 (±3.2)	33.5 (±6.2)	36.5 (±4.9)	30.8 (±3.1)	32.0 (±7.5)	42.5 (±5.0)
	B1	0.4	60.5 (±3.0)	59.0 (±5.7)	42.5 (±2.7)	59.0 (±3.0)	58.0 (±3.9)	54.0 (±5.1)	57.5 (±5.1)	59.0 (±5.0)	57.7 (±2.6)	55.0 (±3.3)	56.0 (±3.3)	61.0 (±5.8)	53.5 (±2.6)	54.5 (±3.5)	66.0 (±7.7)
	B2	0.5	45.8 (±4.0)	44.6 (±4.6)	32.0 (±1.8)	43.5 (±2.7)	43.0 (±2.7)	44.2 (±3.6)	40.5 (±5.1)	41.8 (±2.2)	46.5 (±2.7)	39.5 (±2.0)	41.0 (±5.5)	47.7 (±2.3)	38.2 (±2.7)	39.5 (±4.5)	51.5 (±3.5)
	B3	0.6	37.8 (±4.6)	36.5 (±5.0)	23.8 (±4.3)	33.9 (±2.1)	35.0 (±3.7)	32.5 (±2.7)	31.8 (±2.3)	32.5 (±2.2)	35.5 (±4.4)	29.7 (±3.4)	31.4 (±2.0)	27.5 (±2.5)	29.5 (±4.6)	30.5 (±3.9)	42.0 (±2.5)
PRISMS (100 x 100 x 300 mm)	A1	0.4	61.5 (±2.5)	60.0 (±1.7)	44.8 (±2.1)	60.0 (±6.7)	60.0 (±3.1)	52.8 (±2.7)	58.0 (±4.4)	59.5 (±3.7)	60.5 (±5.0)	57.5 (±2.3)	59.0 (±2.7)	62.5 (±2.5)	56.0 (±4.9)	57.0 (±3.2)	67.5 (±2.1)
	A2	0.5	51.0 (±5.3)	49.5 (±2.5)	35.0 (±3.12)	48.5 (±3.6)	50.0 (±4.8)	44.5 (±2.6)	46.0 (±3.0)	46.7 (±4.8)	48.5 (±2.6)	43.8 (±2.9)	45.1 (±2.4)	51.6 (±3.6)	44.2 (±3.7)	45.0 (±6.4)	55.0 (±2.5)
	A3	0.6	39.6 (±2.1)	38.5 (±4.7)	26.0 (±2.2)	36.5 (±3.2)	37.0 (±5.7)	35.0 (±2.2)	35.0 (±1.5)	36.5 (±3.9)	37.5 (±2.2)	33.0 (±4.9)	35.0 (±5.1)	39.0 (±3.0)	33.5 (±2.5)	35.0 (±6.7)	.5 (±3.9)
	B1	0.4	57.0 (±3.7)	56.5 (±3.6)	43.0 (±4.1)	57.0 (±6.5)	56.5 (±3.5)	53.7 (±1.9)	55.0 (±6.7)	56.0 (±2.4)	56.5 (±4.2)	55.0 (±4.7)	52.0 (±6.4)	60.5 (±4.4)	53.5 (±2.2)	51.5 (±3.9)	64.0 (±3.0)
	B2	0.5	46.8 (±2.4	46.0 (±3.2)	31.5 (±2.6)	45.0 (±3.7)	45.5 (±2.2)	39.0 (±2.9)	44.0 (±2.1)	43.0 (±2.4)	47.5 (±3.1)	43.5 (±2.6)	42.5 (±1.8)	49.5 (±3.6)	41.5 (±6.6)	40.0 (±2.4)	52.0 (±3.3)
	B3	0.6	37.5 (±2.0)	37.0 (±3.9)	23.5 (±2.7)	35.0 (±4.2)	36.0 (±5.2)	33.5 (±3.1)	34.5 (±2.6)	35.5 (±3.2)	37.0 (±3.0)	33.5 (±4.8)	32.5 (±4.1)	38.5 (±3.6)	33.0 (±5.5)	30.8 (±4.7)	40.9 (±2.9)

*95% Confidence limits are given in brackets.

Table (4.8) Compressive Strength of Concrete Cubes, Cylinders, and Prisms at Various Ages, w/c Ratio and Storage Conditions - N/mm^2.

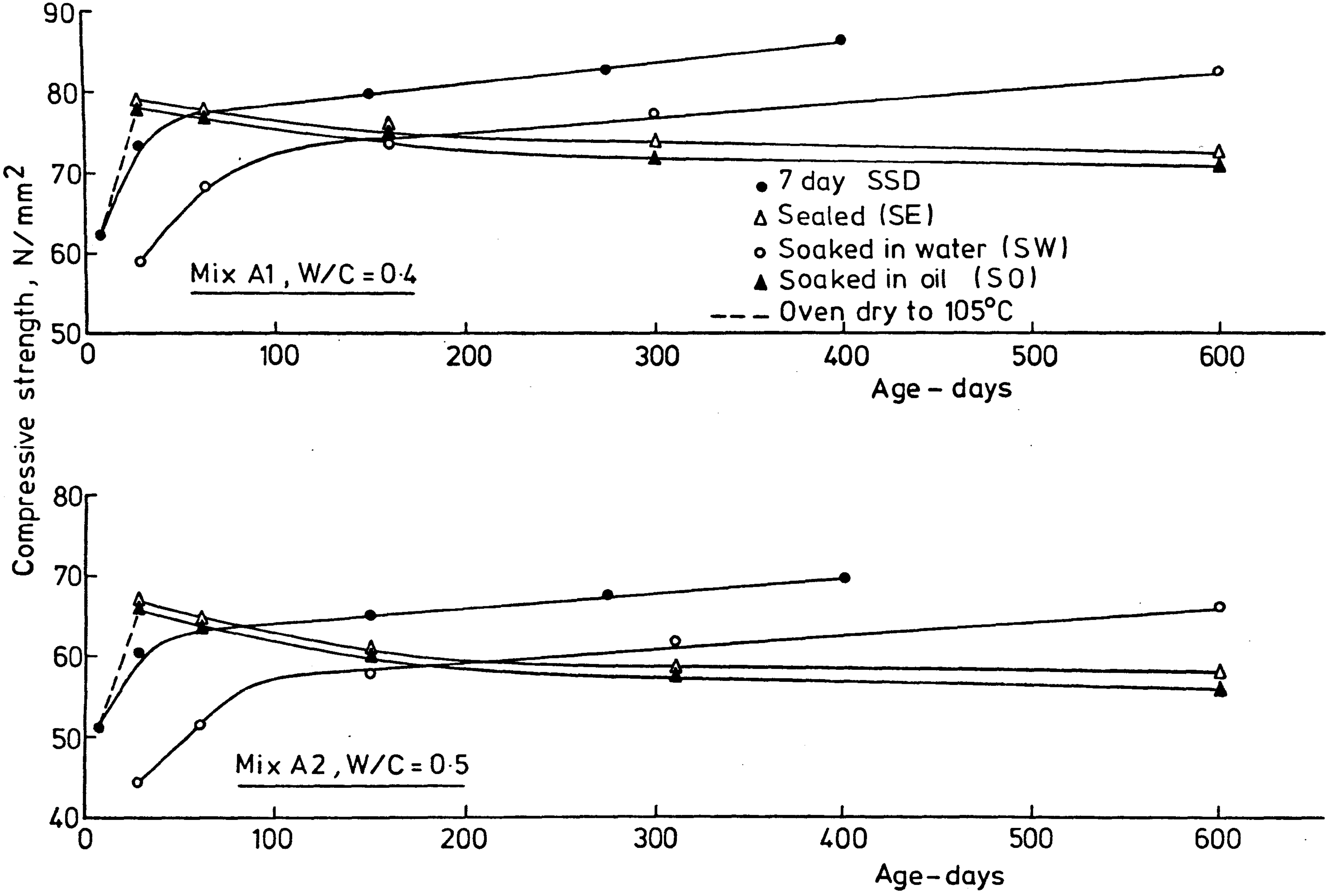


FIG. 4-10 THE EFFECT OF CURING CONDITION ON THE CUBE CRUSHING STRENGTH OF CONCRETE

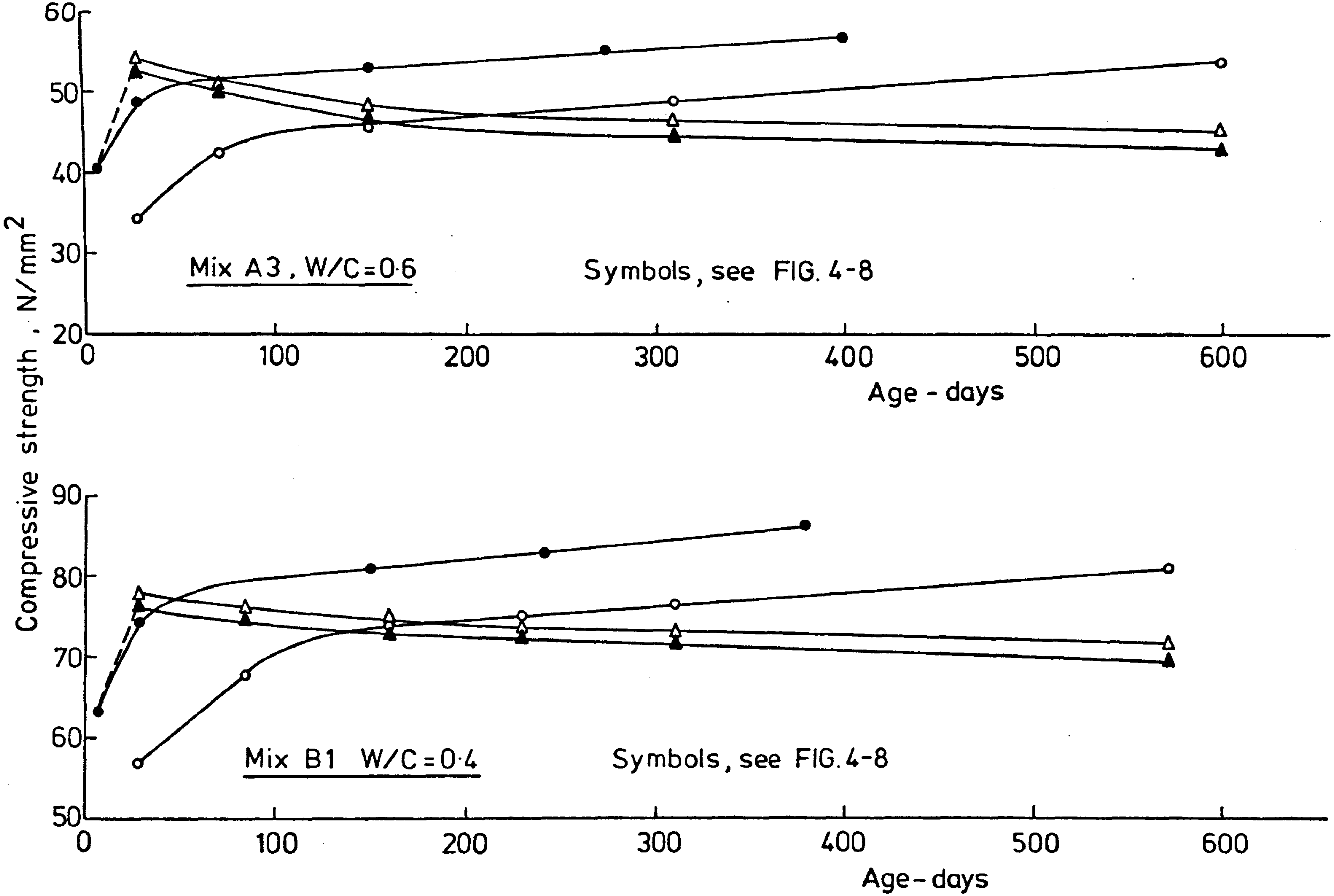


FIG. 4-11 THE EFFECT OF CURING CONDITION ON THE CUBE CRUSHING STRENGTH OF CONCRETE

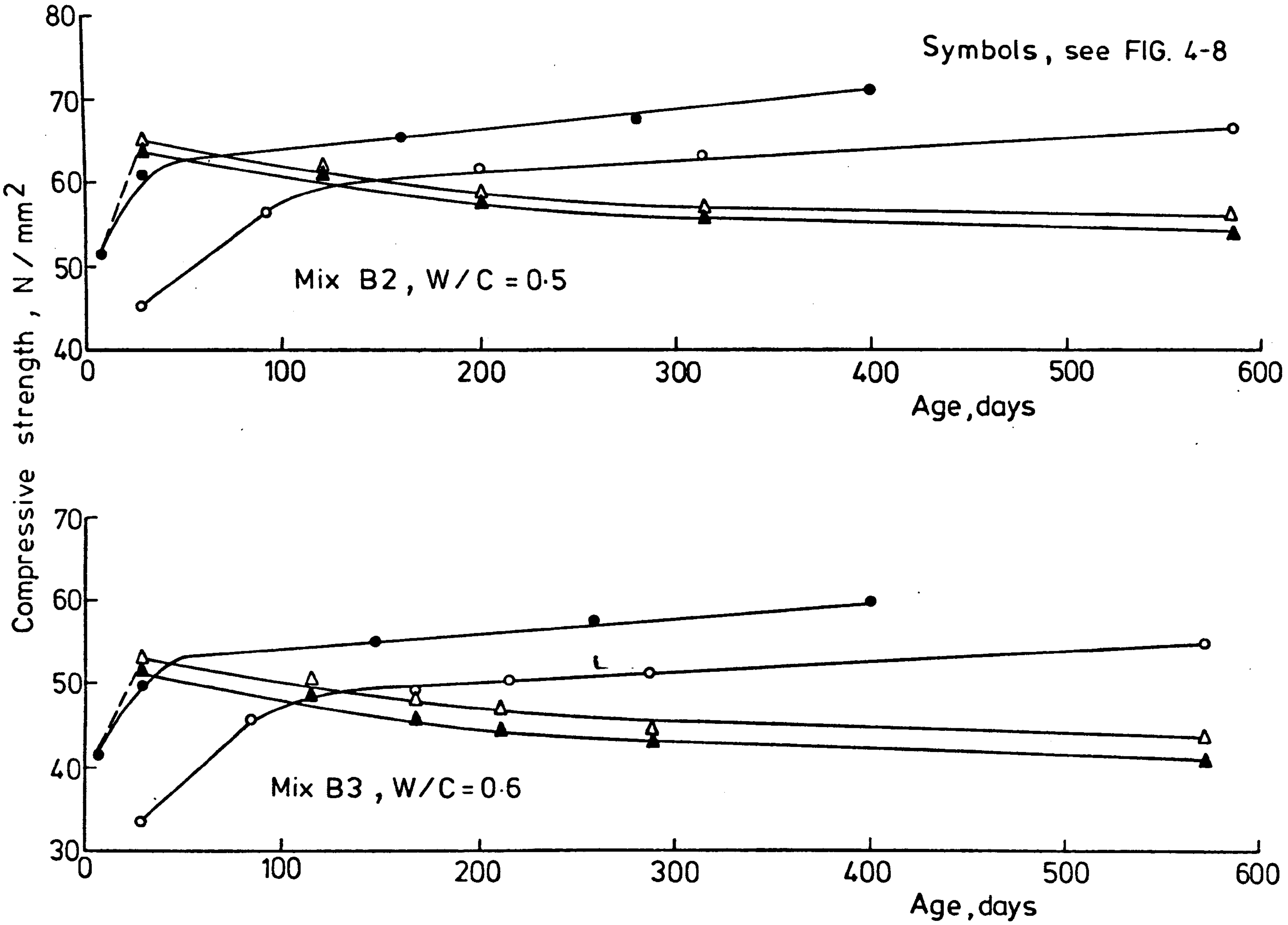


FIG.4-12 THE EFFECT OF CURING CONDITION ON THE CUBE CRUSHING STRENGTH OF CONCRETE

reduction was higher for mixes of higher w/c ratio which had a greater porosity and therefore a larger amount of absorbed water. This sharp reduction was then followed by an initially high rate of strength gain, which gradually decreased with time, but the strength did not reach that of the continuously wet cured concrete. The relationship between the strength of an oven dried specimen resoaked in water $(fc)_t$ and the soaking period for the test results shown in figs. (4.10, 4.11, 4.12), can be expressed exponentially as:

$$(fc)_t = (fc') \, t^{a(w/c)} \qquad \ldots\ldots \qquad (4.4)$$

where:-

fc' = cube crushing strength (N/mm^2) of oven dried specimen soaked in water for one day.

t = soaking period - day.

w/c = water-cement ratio.

a = constant equal to 0.12 for Mix A and 0.14 for Mix B.

The coefficient of correlation for this formula was equal to 97.90 and the maximum error was approximately equal to 9% compared to the experimental results. This error was smaller for concrete of low w/c ratio and was reduced to 3% for the concrete specimens tested after a soaking period of 100 days.

4. There was no immediate sharp reduction of strength when the dry specimens were soaked in crude oil. There was only a very small reduction, which varied between 1.5-2.5% for concrete of Mix A and 2-3% for concrete of Mix B. This reduction is not significant at 0.05 level ($P \leqslant 0.05$). The

reduction in the strength increased as the specimens were kept continuously soaked in oil for longer periods due to the increasing amount of absorbed oil. After a soaking period of 600 days, the reduction in the strength of oil soaked specimens was significant ($P \leqslant 0.05$) and varied between 9.5-20% for concrete of mix A (with an oil content variation of 5-5.65% by weight) and 10.5-22.5% for concrete of Mix B (with an oil content variation of 4.65-5.45% by weight), as a percentage of the dry concrete strength before soaking in oil. The percentage reduction was higher in the specimens of higher w/c ratio due to the high amount of absorbed oil. Using the test results presented in figs. (4.10, 4.11, 4.12), the strength of oil soaked concrete cube at any soaking period can be expressed exponentially by the following formula:-

$$(fc)_t = (fc)_d \, t^{a(w/c)} \qquad ..... \qquad (4.5)$$

where:-

$(fc)_d$ = cube crushing strength (N/mm^2) of oven dried specimen before soaking in oil.

t = soaking period (days) in oil.

w/c = water-cement ratio.

a = constant equal to (-0.05) for Mix A and (-0.055) for Mix B.

The maximum error obtained by this formula was equal to 5.5% compared to the experimental results. This error reduced as the soaking period increased beyond 150 days.

4.4.1.2 The Effect of Oil Saturation on the Cube Strength of Concrete at Various w/c Ratios.

The values of the compressive strength of concrete for both mixes tested at different storage conditions are plotted in fig. (4.13) against the corresponding w/c ratio. The following observations are made:

1. The relationship between cube crushing strength and w/c ratio for the curves presented in fig. (4.13), for any storage condition, can be expressed in the following exponential form:-

$$fc = a\,e^{b(w/c)} \quad \ldots\ldots \quad (4.6)$$

The constants a and b are given in table (4.9).

2. The compressive strength of cubes soaked in oil for 600 days was lower than both water saturated and sealed cubes tested at the same age. The percentage reduction in strength for concretes made with larger size of aggregates was higher than that with smaller size of aggregates. The percentage of reduction due to saturation with oil was between 1.5-5.5 for 10 mm aggregate concrete (Mix A) and 3.5-6.8% for 20 mm aggregate concrete (Mix B), as a percentage of the corresponding sealed specimens.

Mix Designation	Curing Condition		a	b	r
A	600-days	SE	189	-2.39	99.91
	"	SO	200	-2.57	99.74
	"	SW	200	-2.21	99.91
B	600-days	SE	194	-2.46	99.80
	"	SO	200	-2.64	99.92
	"	SW	176	-1.94	99.95

Table (4.9) Constants of Regression - equn. (4.6) Compressive Strength Vs w/c Ratio.

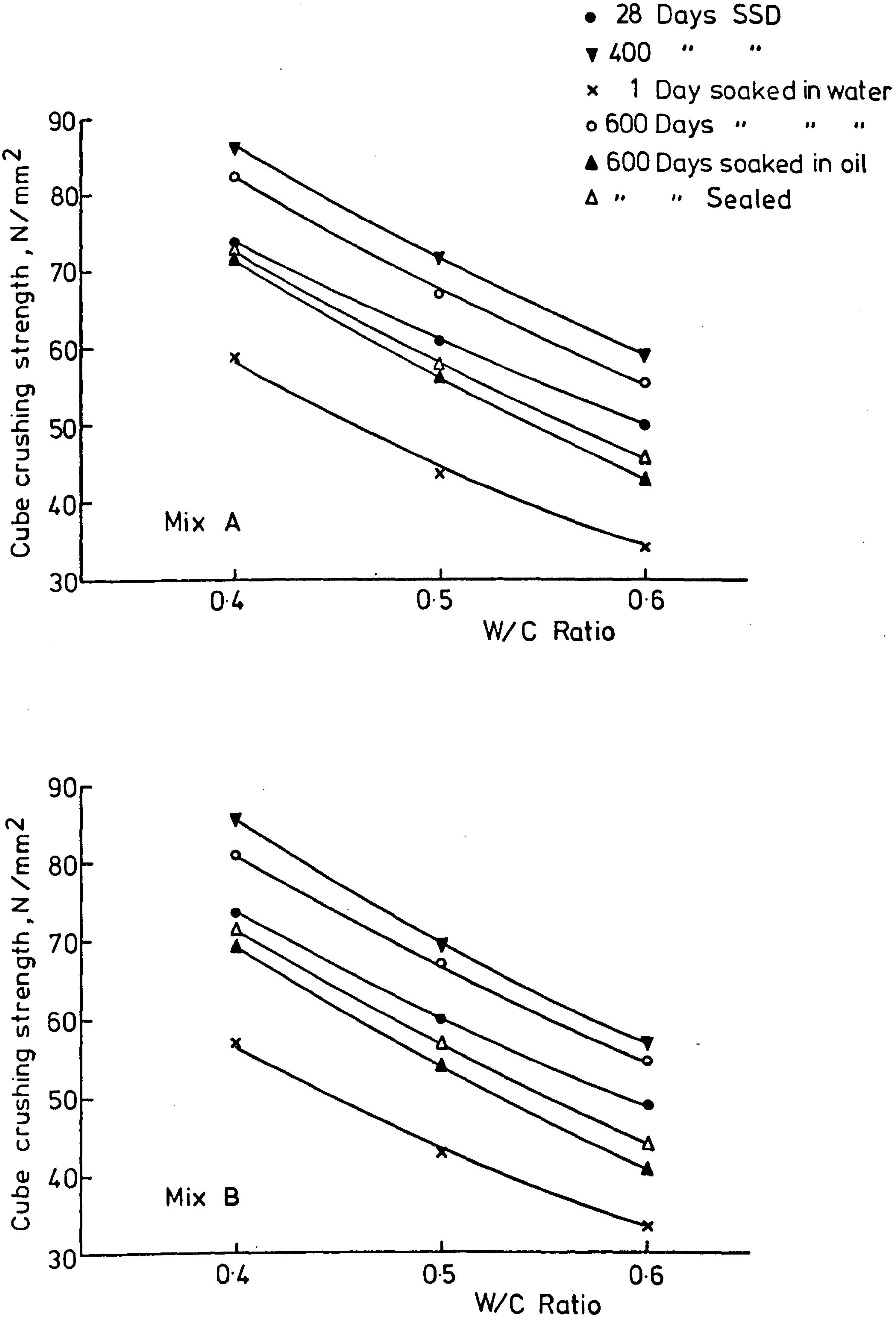


FIG. 4-13 THE EFFECT OF STORAGE CONDITION AND W/C RATIO ON THE CUBE CRUSHING STRENGTH OF CONCRETE

4.4.1.3 Effect of Oil Saturation on the Cylinder and Prism Compressive Strength.

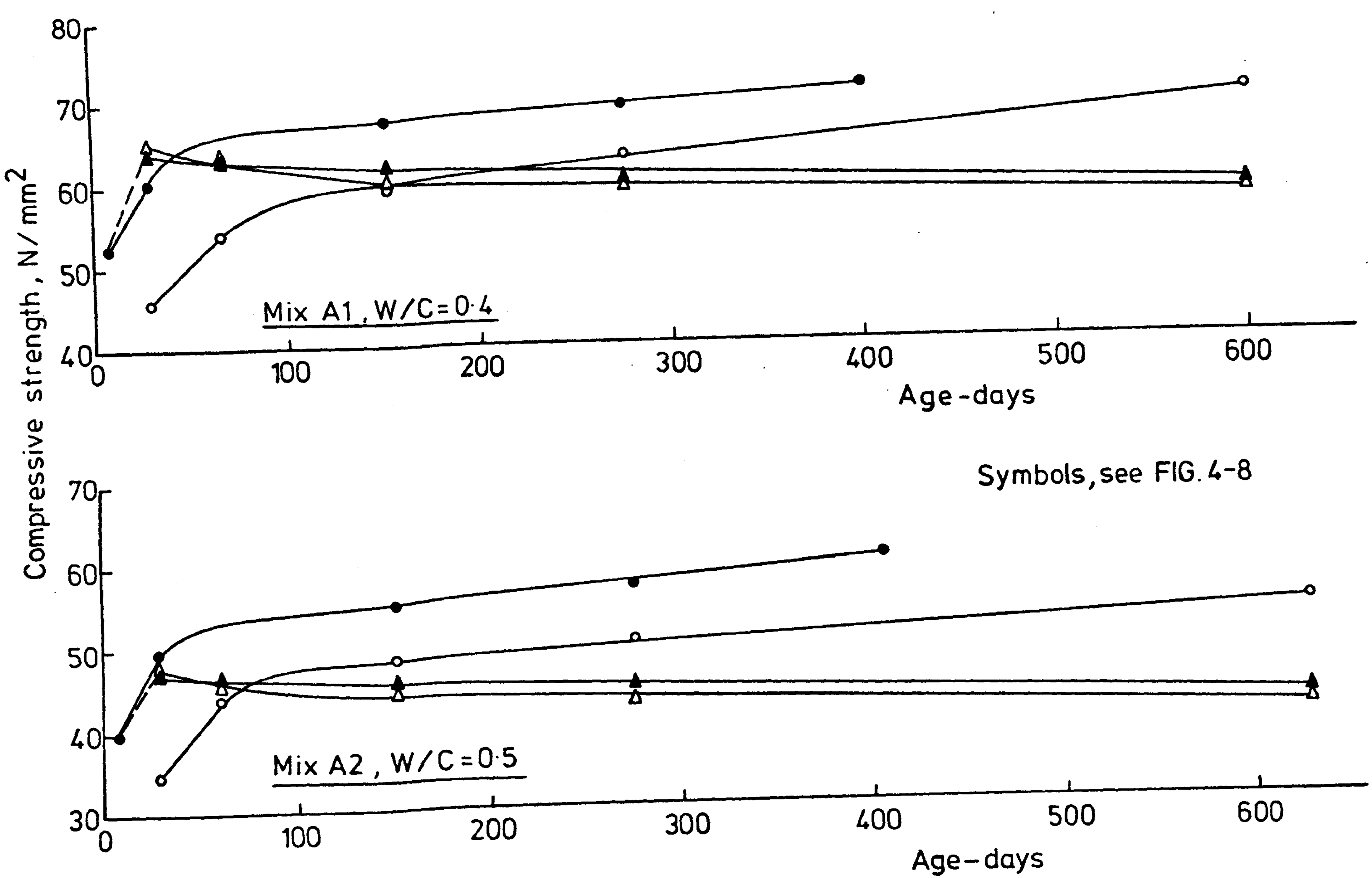
The specimens were cylinders (100x200 mm) and prisms (100x100x300 mm) having the same mix proportions and curing condition as those used for the cube test. The test results are shown in tables (4.7) and (4.8) and illustrated in figs. (4.14 to 4.19). The following observations are made:

1. The cylinder and the prism compressive strength after being oven dried at 105° C for 14 days to constant weight increased significantly ($P \leqslant 0.05$) between 22-30% and 18-28% respectively. All results are given as a percentage of the 7-day moist cured strength. The increase in the compressive strength reduced as the specimen size increased.

2. The specimens kept continuously cured in water for 400 days after demoulding and tested in a saturated surface dry condition increased in strength by between 20-24.5% for cylinder specimens and between 24-30% for prism specimens, as a percentage of the 28-day moist cured strength.

3. The 600 day compressive strength of the sealed concrete specimens reduced in strength by between 12.5-22% for cylinder specimens and 8-14% for prism specimens, as a percentage of the 1-day reference strength. The reduction for the concrete of larger size aggregate (Mix B) was between 1-5% higher than that for concrete of smaller size aggregate (Mix A).

4. The cylinder compressive strength fell by between 29-34% for Mix A and 30-37% for Mix B as a percentage of the 1-day reference strength after 24 hours soaking in water. The prism



Symbols, see FIG. 4-8

FIG. 4-14 THE EFFECT OF CURING CONDITION ON THE COMPRESSIVE STRENGTH OF CONCRETE CYLINDER

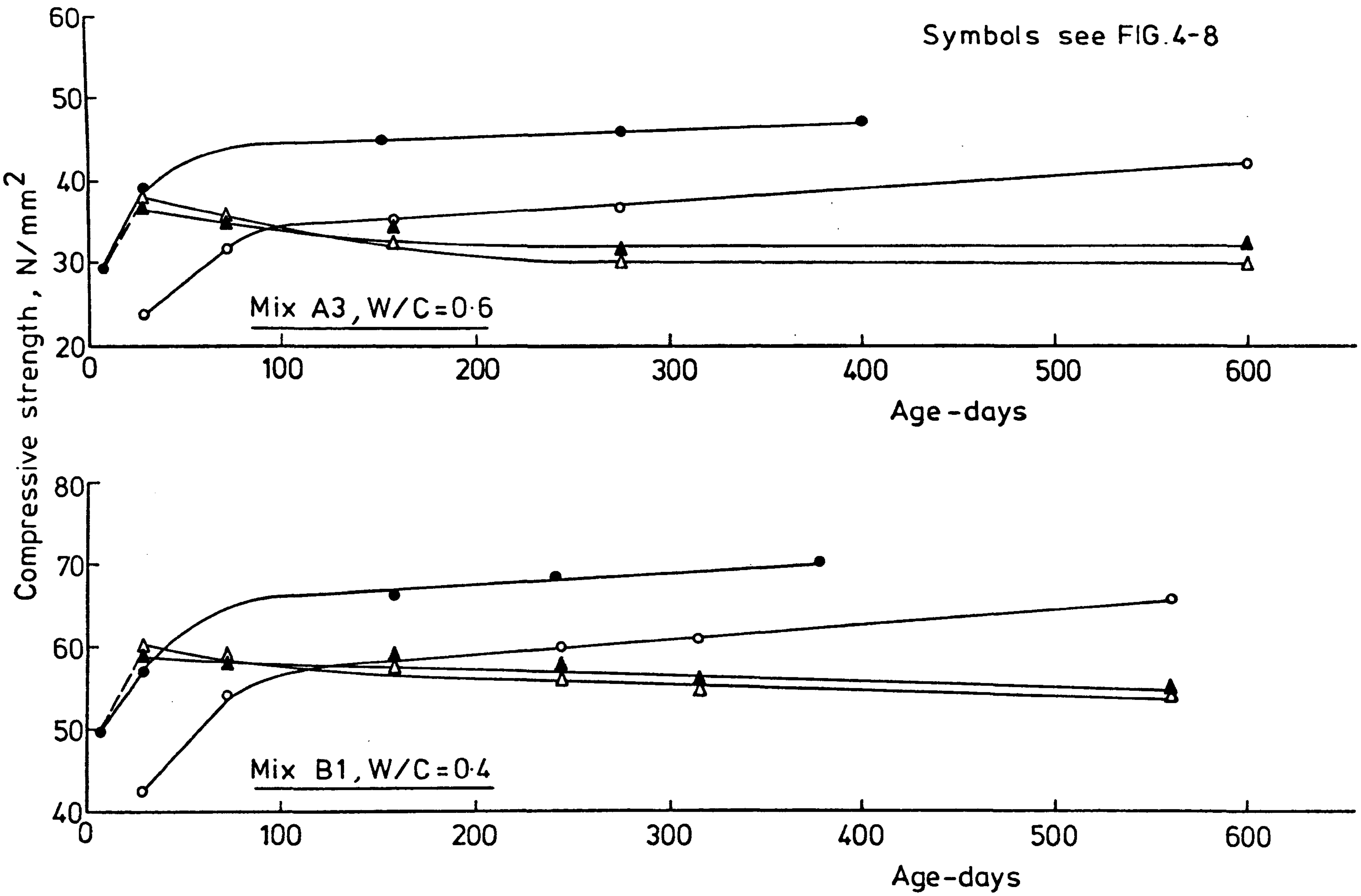


FIG. 4-15 THE EFFECT OF CURING CONDITION ON THE COMPRESSIVE STRENGTH OF CONCRETE CYLINDER

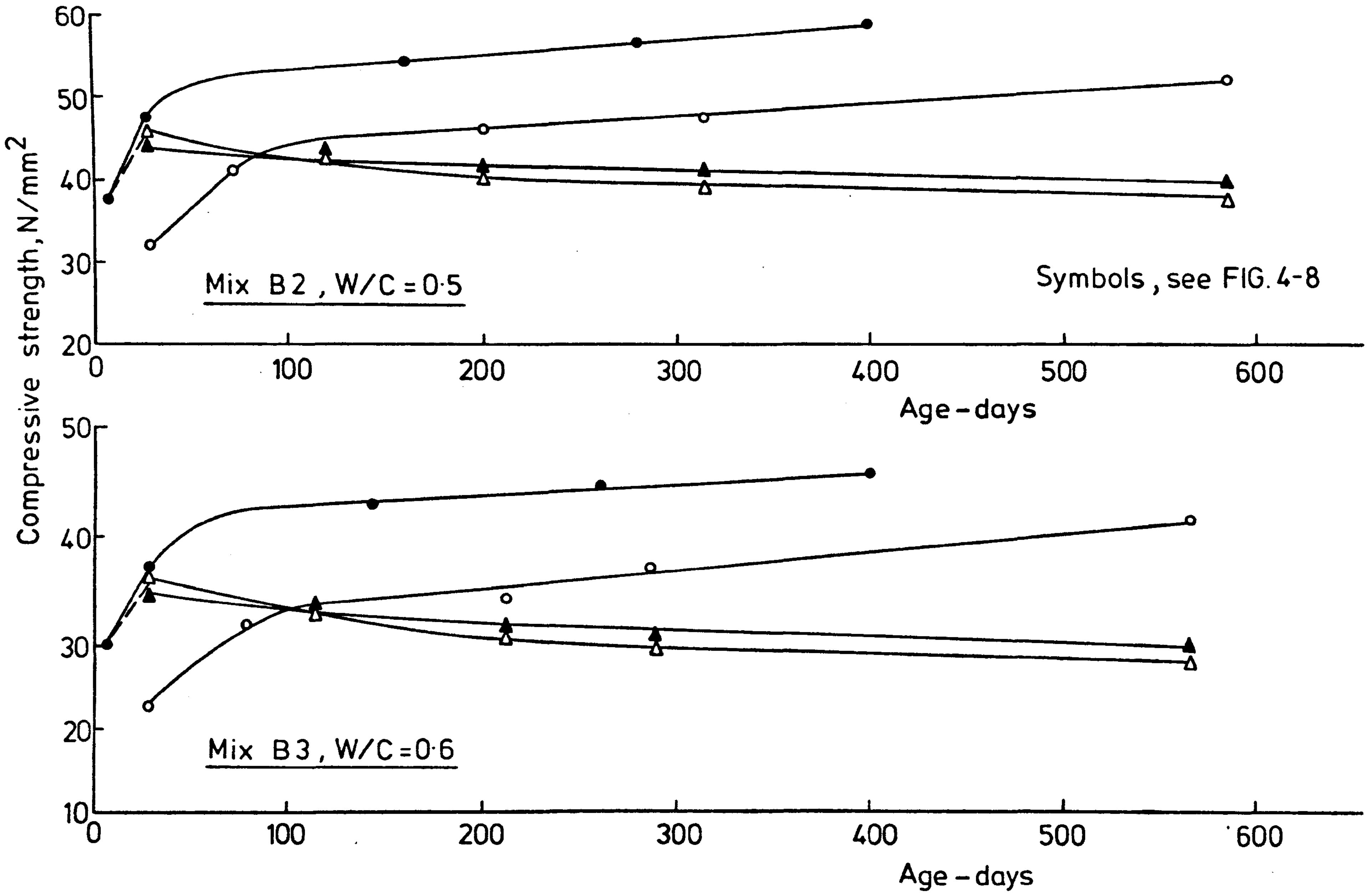


FIG. 4-16 THE EFFECT OF CURING CONDITION ON THE COMPRESSIVE STRENGTH OF CONCRETE CYLINDER

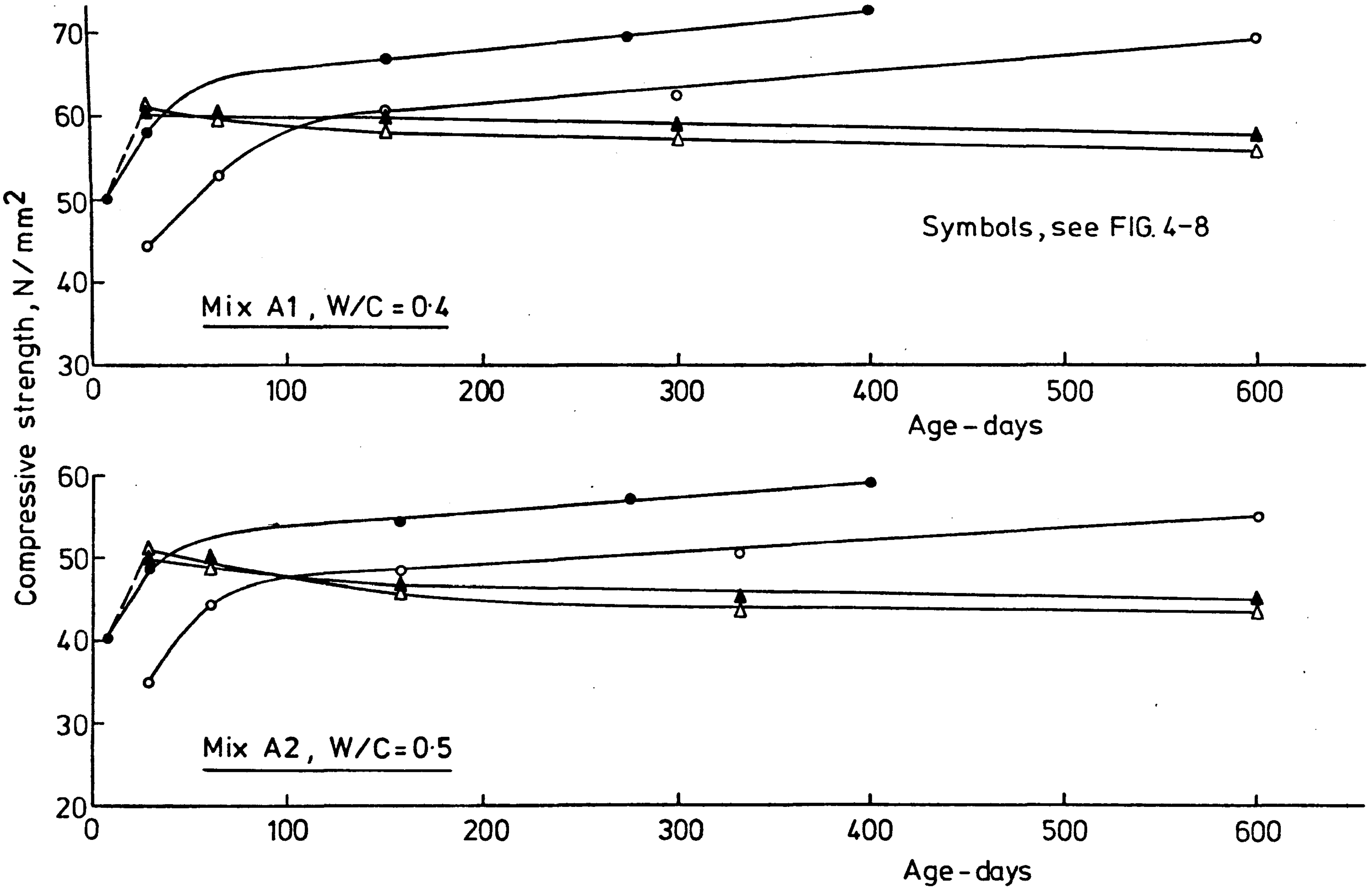
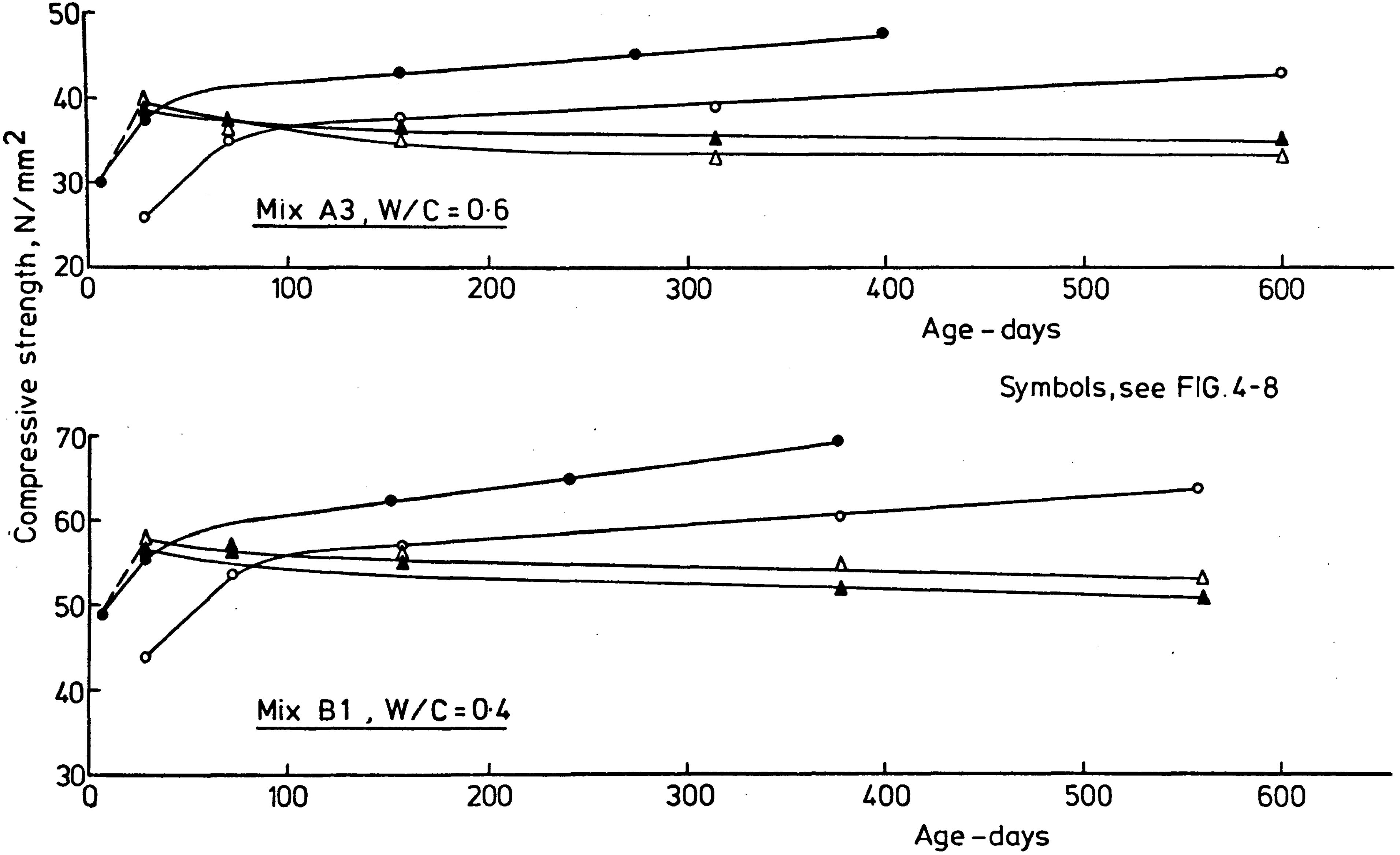


FIG. 4-17. THE EFFECT OF CURING CONDITION ON THE COMPRESSIVE STRENGTH OF CONCRETE PRISMS



Symbols, see FIG. 4-8

FIG. 4-18 THE EFFECT OF CURING CONDITION ON THE COMPRESSIVE STRENGTH OF CONCRETE PRISMS

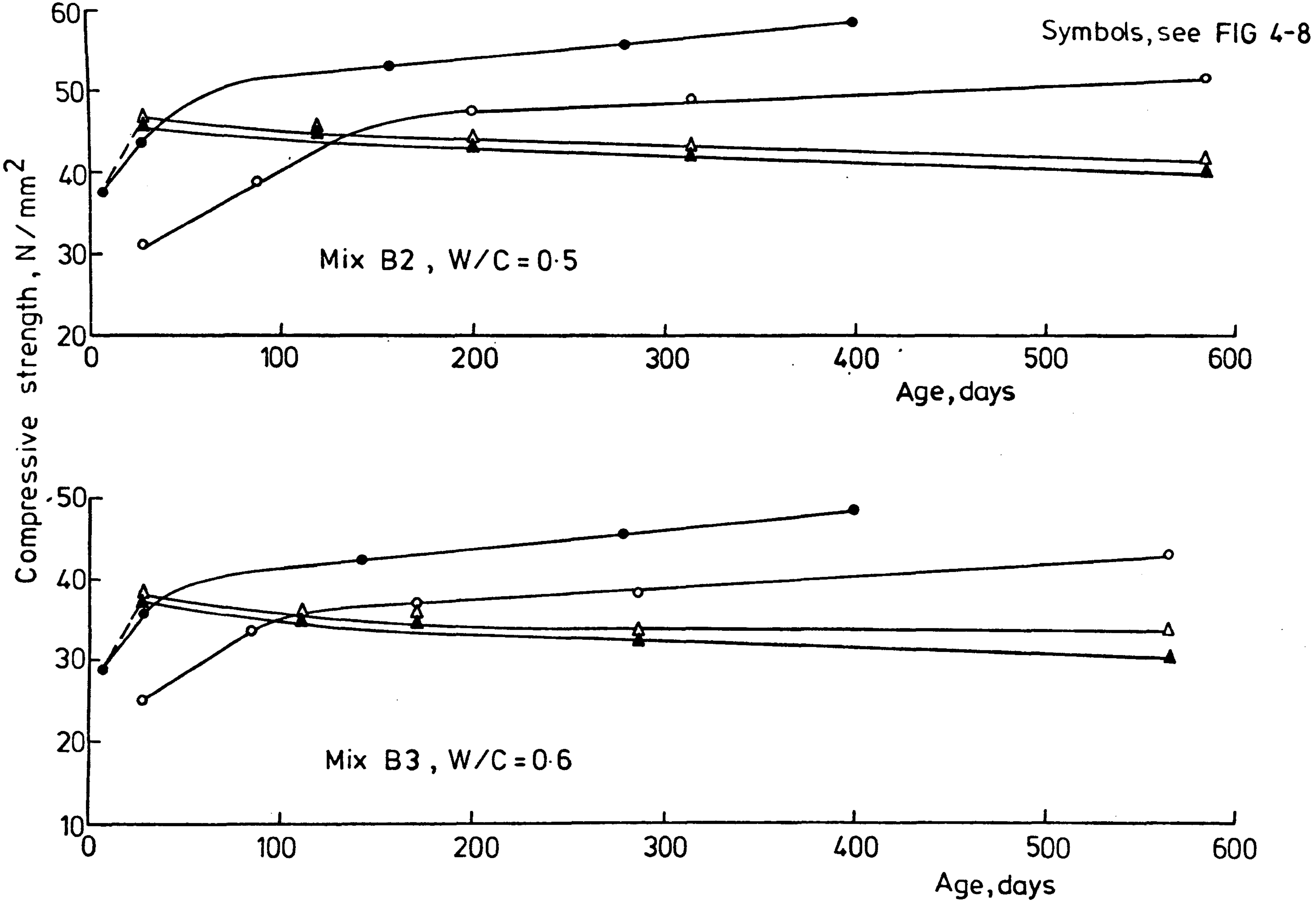


FIG. 4-19 THE EFFECT OF CURING CONDITION ON THE COMPRESSIVE STRENGTH OF CONCRETE PRISMS

strength fell by between 26-33% for concrete of Mix A and 26-38% for concrete of Mix B, as a percentage of the 1-day reference strength after 24 hours soaking in water. This sharp reduction was then followed by an initially high rate of strength gain, which gradually decreased with time. After water saturation for 600 days, the cylinder and the prism compressive strength was higher than that of the 28-day moist cured strength by about 8-16% and 12-17.5% for mixes A and B respectively.

5. The reduction in the cylinder strength after being soaked in oil for 600 days varied between 10-15% for concrete of Mix A (with an oil content variation of 5-6.1% by weight) and 10-19.5% for concrete of Mix B (with an oil content variation of 4.61-5.18% by weight). Using concrete prisms, the reduction varied between 8-14% for concrete of Mix A (with an oil content variation of 5.1-5.92% by weight) and 10-18% for concrete of Mix B (with an oil content variation of 4.51-5.1% by weight). All results are given as a percentage of the dry strength before soaking in oil. The percentage of reduction was greater in the specimens of Mix B due to the presence of larger aggregates and therefore weaker aggregate-cement bond specially after being dried in the oven, cooled, and then soaked in oil as discussed later in section (4.5.2).

4.4.2 Effect of Oil Saturation on the Splitting Tensile Strength of Concrete.

The splitting tensile strength was calculated using the formula given in B.S. No. 1881: 1970, and the results shown in table (4.10) are the average of three values. The results are plotted against age as shown in fig. (4.20) for both Mixes A and B. From the test results the following observations can be made:

1. The tensile splitting strength increased by about 2% for Mix A and 2.5% for Mix B for the 14 days 105° C oven dried concrete, both percentages being calculated as a percentage of the 7-day moist cured strength. This increase was not significant at the 0.05 level. The splitting tensile strength of the sealed specimens decreased significantly ($P \leqslant 0.05$) with time. This reduction was between 6-10% for Mix A and 8-12% for Mix B within a storage period of 450 days.

2. The strength of dried specimens decreased significantly between 18-20% for Mix A and 20-22% for Mix B after 24 hours of water soaking. After this sharp reduction, the strength began to increase and after a soaking period of 450 days, the splitting tensile strength was 4.5 - 6% higher than that of the dried specimens before soaking in water.

3. The strength of the oven dried specimens increased by 4 - 6% for concrete of Mix A and 3 - 4.5% for concrete of Mix B after being soaked in oil for 24 hours. This increase was not significant at the 0.05 level. But as the soaking period increased, the splitting tensile strength for both mixes and for all w/c ratios decreased significantly ($P \leqslant 0.05$) with time. The reduction was between 4-8% for specimens of Mix A (with an oil content variation of 4.8-5.47% by weight) and 6-9.5% for Mix B (with an oil content variation of 4.65-5.1 by weight) within a soaking period of 450 days compared with that of the 1-day oil soaked strength.

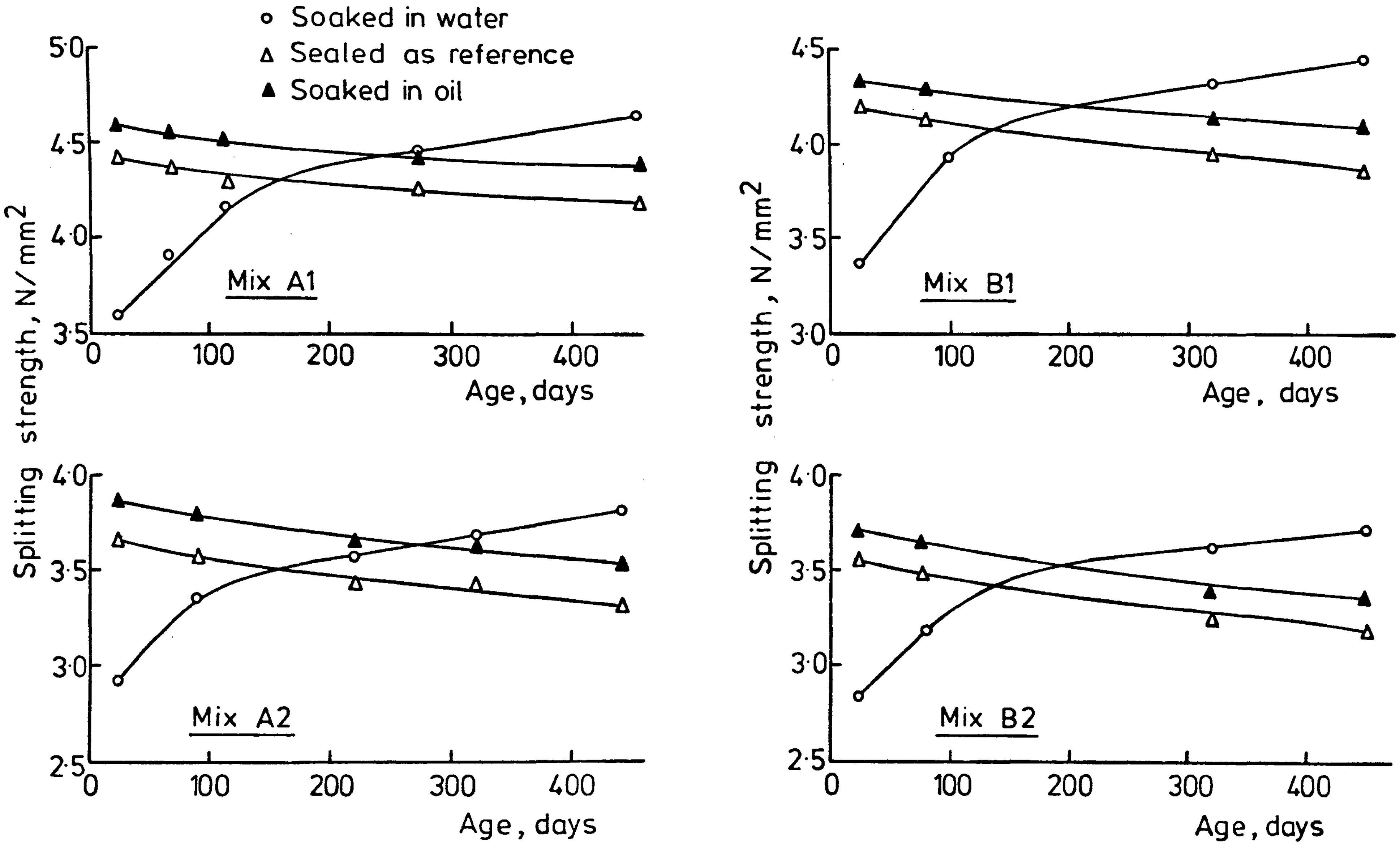


FIG. 4-20 THE EFFECT OF STORAGE CONDITION ON THE TENSILE SPLITTING STRENGTH OF CONCRETE OF VARIOUS MIX PROPORTIONS

4.4.3 Effect of Oil Saturation on the Direct Tensile Strength of Concrete.

The uniaxial tensile strength of oil soaked plain concrete beams is compared with that of other specimens stored under conditions previously mentioned (Section 3.6). The values of the direct tensile strength shown in table (4.11) were the mean of five results. Variation of individual sample strength from the mean value was large with a maximum difference from the mean of about 16% and an average difference of about 10% The test results given in table (4.11) and fig. (4.21) suggest the following observations:

1. The direct tensile strength of the sealed concrete specimens decreased significantly ($P \leqslant 0.05$) by about 8% for concrete of Mix A_1 ($w/c = 0.4$) and 12% for Mix A_2 ($w/c = 0.5$) after storing for 455 days in the laboratory, both percentages being calculated as a percentage of the 1-day dry concrete specimen.
2. The strength of dried specimens shows significant ($P \leqslant 0.05$) decrease by about 14% for Mix A_1 and 20% for Mix A_2 after 50 days of soaking in water as a percentage of the dry strength before soaking in water. After a soaking period of 440 days, the direct tensile strength was between 9-12% higher than that of the dried specimens before soaking in water.
3. The strength of oil soaked specimens increased by about 10.5 - 11.5% after 50 days of soaking in oil. As the soaking period increased, the uniaxial tensile strength for both mixes decreased significantly ($P \leqslant 0.05$) with time due to the increasing amount of absorbed oil. The reduction was about

Mix Designation	w/c Ratio	7-day moist curing	1-day			60-day			300-day			470-day		
			Sealed as Reference (SE)	Soaked in oil (SO)	Soaked in water (SW)	Sealed as Reference (SE)	Soaked in oil (SO)	Soaked in water (SW)	Sealed as Reference (SE)	Soaked in oil (SO)	Soaked in water (SW)	Sealed as Reference (SE)	Soaked in oil (SO)	Soaked in water (SW)
A1	0.4	4.33* (±0.33)	4.42 (±0.28)	4.59 (±0.20)	3.60 (±0.25)	4.36 (±0.28)	4.56 (±0.37)	3.91 (±0.39)	4.26 (±0.38)	4.43 (±0.34)	4.46 (±0.33)	4.18 (±0.26)	4.39 (±0.26)	4.65 (±0.50)
A2	0.5	3.57 (±0.23)	3.65 (±0.26)	3.86 (±0.25)	2.93 (±0.15)	3.56 (±0.16)	3.79 (±0.36)	3.36 (±0.37)	3.41 (±0.12)	3.63 (±0.29)	3.69 (±0.33)	3.31 (±0.30)	3.54 (±0.30)	3.81 (±0.40)
B1	0.4	4.11 (±0.34)	4.33 (±0.34)	3.36 (±0.32)	4.13 (±0.51)	4.29 (±0.36)	3.91 (±0.49)	3.44 (±0.40)	4.14 (±0.29)	4.31 (±0.31)	3.86 (±0.30)	4.08 (±0.30)	4.08 (± .24)	4.45 (±0.36)
B2	0.5	3.55 (±0.25)	3.70 (±0.34)	2.81 (±0.30)	3.49 (±0.20)	3.64 (±0.40)	3.19 (±0.22)	3.24 (±0.44)	3.39 (±0.39)	3.61 (±0.49)	3.19 (±0.37)	3.35 (±0.31)	3.35 (±0.36)	3.71 (±0.31)

*95% Confidence limits are given in brackets.

Table (4.10) The Splitting Tensile Strength of concrete at various ages, w/c ratios, and storage conditions - N/mm^2.

Type of Test	Mix Designation	w/c Ratio	50-day			200-day			300-day			455-day		
			Sealed as Reference (SE)	Soaked in oil (SO)	Soaked in water (SW)	Sealed as Reference (SE)	Soaked in oil (SO)	Soaked in water (SW)	Sealed as Reference (SE)	Soaked in oil (SO)	Soaked in water (SW)	Sealed as Reference (SE)	Soaked in oil (SO)	Soaked in water (SW)
Uniaxial Tensile Strength	A1	0.4	3.68* (±0.29)	4.10 (±0.35)	3.15 (±0.30)	3.50 (±0.42)	3.83 (±0.32)	3.68 (±0.29)	3.40 (±0.35)	3.78 (±0.19)	3.93 (±0.49)	3.37 (±0.27)	3.74 (±0.30)	4.10 (±0.44)
	A2	0.5	2.88 (±0.26)	3.18 (±0.30)	2.28 (±0.37)	2.60 (±0.20)	2.93 (±0.45)	2.68 (±0.37)	2.58 (±0.45)	2.93 (±0.30)	2.88 (±0.21)	2.53 (±0.35)	2.90 (±0.26)	3.15 (±0.26)
Percentage of absorption	A1	0.4	0.37	3.94	5.70	0.87	4.41	6.62	1.13	4.56	7.17	1.26	4.78	7.32
	A2	0.5	0.57	4.25	5.85	1.03	4.76	6.80	1.43	5.13	7.75	1.51	5.50	8.33

*95% Confidence limits given in brackets.

Table (4.11) The Uniaxial Tensile Strength of Concrete and the Percentage of Absorption at Various Ages, w/c ratio, and Storage Conditions, N/mm^2.

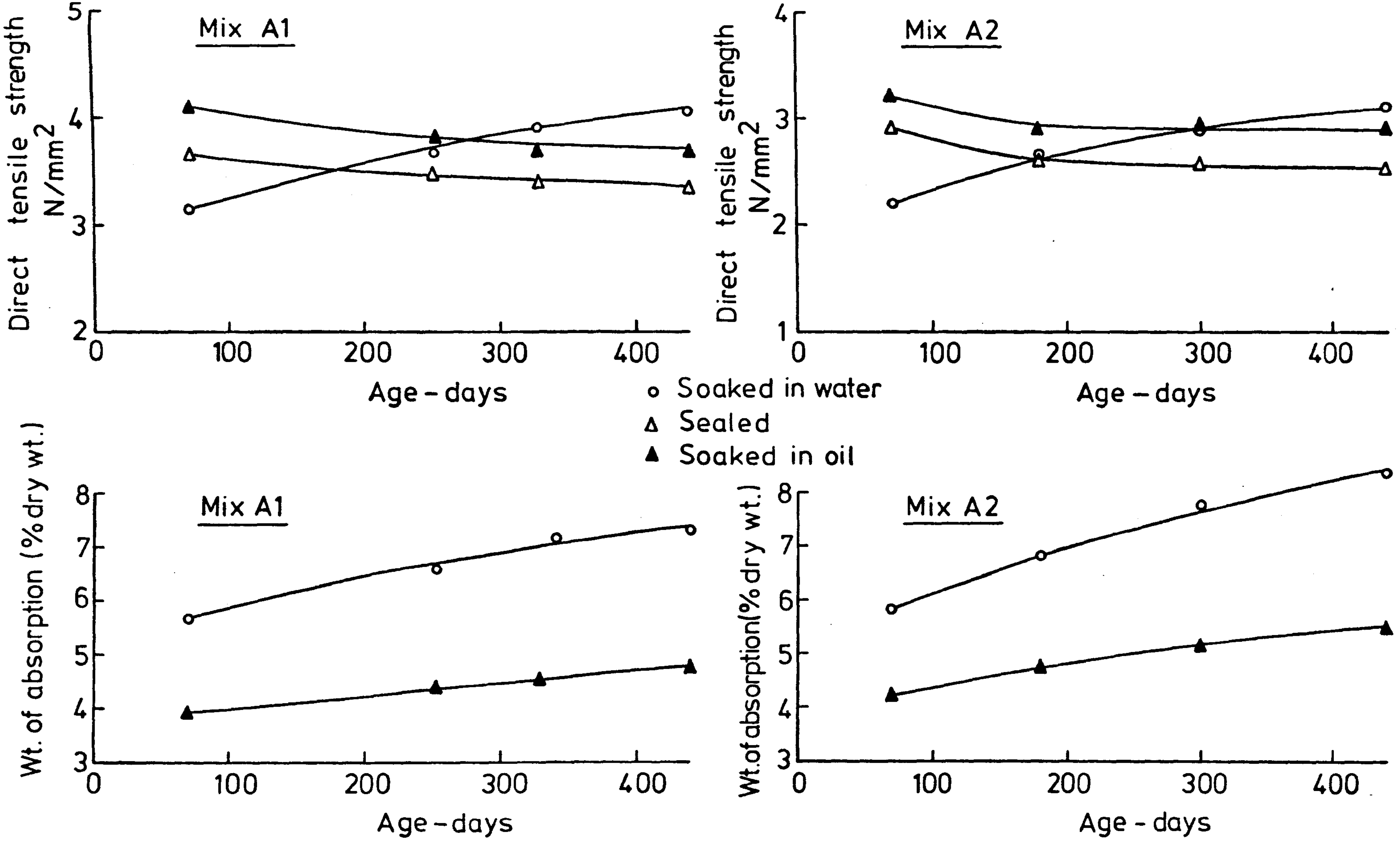


FIG. 4-21 THE EFFECT OF OIL SATURATION ON THE DIRECT TENSILE STRENGTH OF CONCRETE AT VARIOUS AGES AND W/C RATIOS

8.5% for Mix A_1 (with an oil content variation of 4.78% by weight) and 9% for Mix A_2 (with an oil content variation of 5.5% by weight) within a soaking period of 440 days compared with that of the 50 days oil soaked strength.

4.4.4 The Effect of Oil Saturation on the Modulus of Rupture of Concrete.

In the flexural test, the maximum theoretical tensile stress reached at the bottom fibre of the tested beam is the modulus of rupture. Its value was computed from the bending formula given in B.S.No. 1881:1970. Three specimens were tested at each age and the variation of individual sample strength from the mean value was small with an average difference of about 5%. According to the results given in table (4.12) and figs. (4.22,4.23), the following observations are made.

1. The modulus of rupture for both mixes increased by about 15-18% after being dried for 14 days in the oven at 105° C, as a percentage of the 7-day moist cured strength. For all concrete mixes, the actual numerical increase was almost similar and independent of aggregate size.
2. The modulus of rupture of the sealed specimens decreased with time. The largest decrease was between 2.5-5% for Mix A and 4-6.5% for Mix B after being stored for 600 days in the laboratory. This reduction was significant ($P < 0.05$) for all except the 10 mm aggregate concrete of 0.4 w/c ratio.
3. The strength of dried specimens decreased between 24-30% for Mix A and 26-32% for Mix B after 24 hours of water soaking, both percentages being calculated as a percentage of the dry strength before soaking in water. After this sharp

Mix Designation	w/c Ratio	7-day moist curing strength	1 - day			70 - day			150 - day			225 - day			600 - day		
			Sealed (SE)	Soaked in oil (SO)	Soaked in water (SW)	Sealed (SE)	Soaked in oil (SO)	Soaked in water (SW)	Sealed (SE)	Soaked in oil (SO)	Soaked in water (SW)	Sealed (SE)	Soaked in oil (SO)	Soaked in water (SW)	Sealed (SE)	Soaked in oil (SO)	Soaked in water (SW)
A	0.4	6.0* (±.26)	6.9* (±.24)	7.26 (±.31)	5.28 (±.38)	6.88 (±.24)	7.18 (±.49)	5.78 (±.15)	6.85 (±.24)	7.15 (±.36)	5.98 (±.34)	6.75 (±.24)	7.10 (±.25)	6.30 (±.26)	6.73 (±.18)	7.00 (±.26)	6.55 (±.24)
	0.5	5.68 (±.29)	6.57 (±.28)	6.90 (±.25)	4.78 (±.16)	6.43 (±.26)	6.75 (±.27)	5.18 (±.31)	6.35 (±.32)	6.60 (±.24)	5.48 (±.36)	6.28 (±.14)	6.63 (±.15)	5.70 (±.33)	6.30 (±.31)	6.60 (±.28)	6.00 (±.27)
	0.6	5.20 (±.28)	6.08 (±.30)	6.45 (±.26)	4.23 (±.15)	5.98 (±.30)	6.33 (±.22)	4.80 (±.34)	5.93 (±.34)	6.28 (±.35)	5.10 (±.40)	5.85 (±.20)	6.18 (±.24)	5.28 (±.16)	5.78 (±.24)	6.03 (±.30)	5.45 (±.38)
B	0.4	5.68 (±.24)	6.70 (±.27)	6.97 (±.30)	4.95 (±.36)	6.55 (±.28)	6.80 (±.37)	5.45 (±.26)	6.50 (±.27)	6.78 (±.25)	5.65 (±.30)	6.45 (±.31)	6.73 (±.39)	5.90 (±.37)	6.43 (±.25)	6.66 (±.29)	6.13 (±.38)
	0.5	5.27 (±.25)	6.08 (±.22)	6.38 (±.24)	4.23 (±.28)	6.08 (±.28)	6.33 (±.41)	4.68 (±.23)	6.00 (±.29)	6.25 (±.38)	5.15 (±.31)	5.93 (±.23)	6.23 (±.49)	5.33 (±.36)	5.80 (±.25)	6.08 (±.29)	5.48 (±.23)
	0.6	4.78 (±.15)	5.55 (±.20)	5.88 (±.22)	3.78 (±.17)	5.45 (±.12)	5.73 (±.30)	4.33 (±.28)	5.33 (±.34)	5.63 (±.28)	4.63 (±.22)	5.28 (±.41)	5.58 (±.30)	4.73 (±.33)	5.18 (±.31)	5.43 (±.31)	4.80 (±.30)

*95% Confidence limits are given in brackets.

Table (4.12) Modulus of rupture of concrete at various ages, w/c ratio, and curing conditions - N/mm^2.

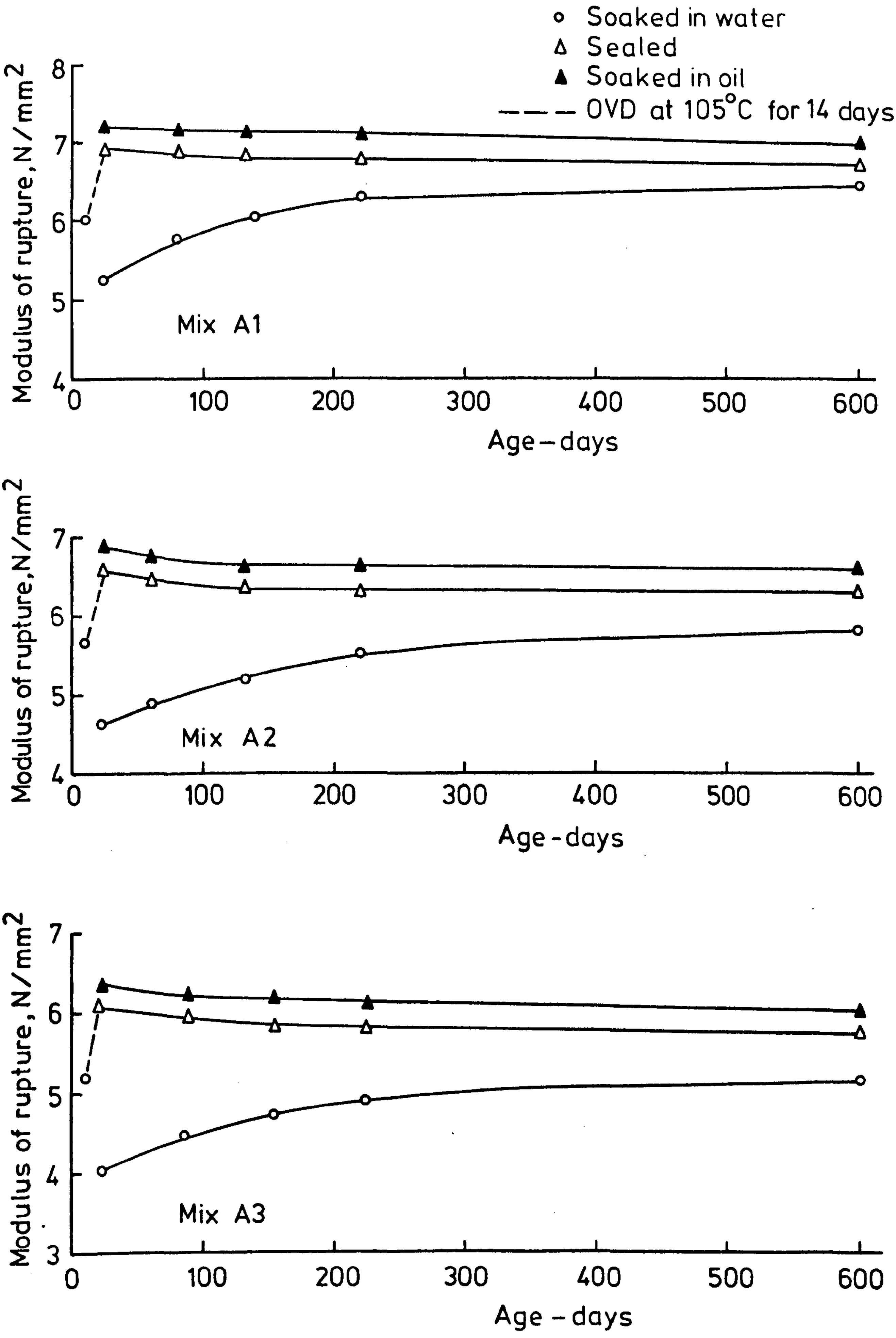


FIG. 4-22 THE EFFECT OF CURING CONDITION ON THE FLEXURAL STRENGTH OF CONCRETE AT VARIOUS AGES

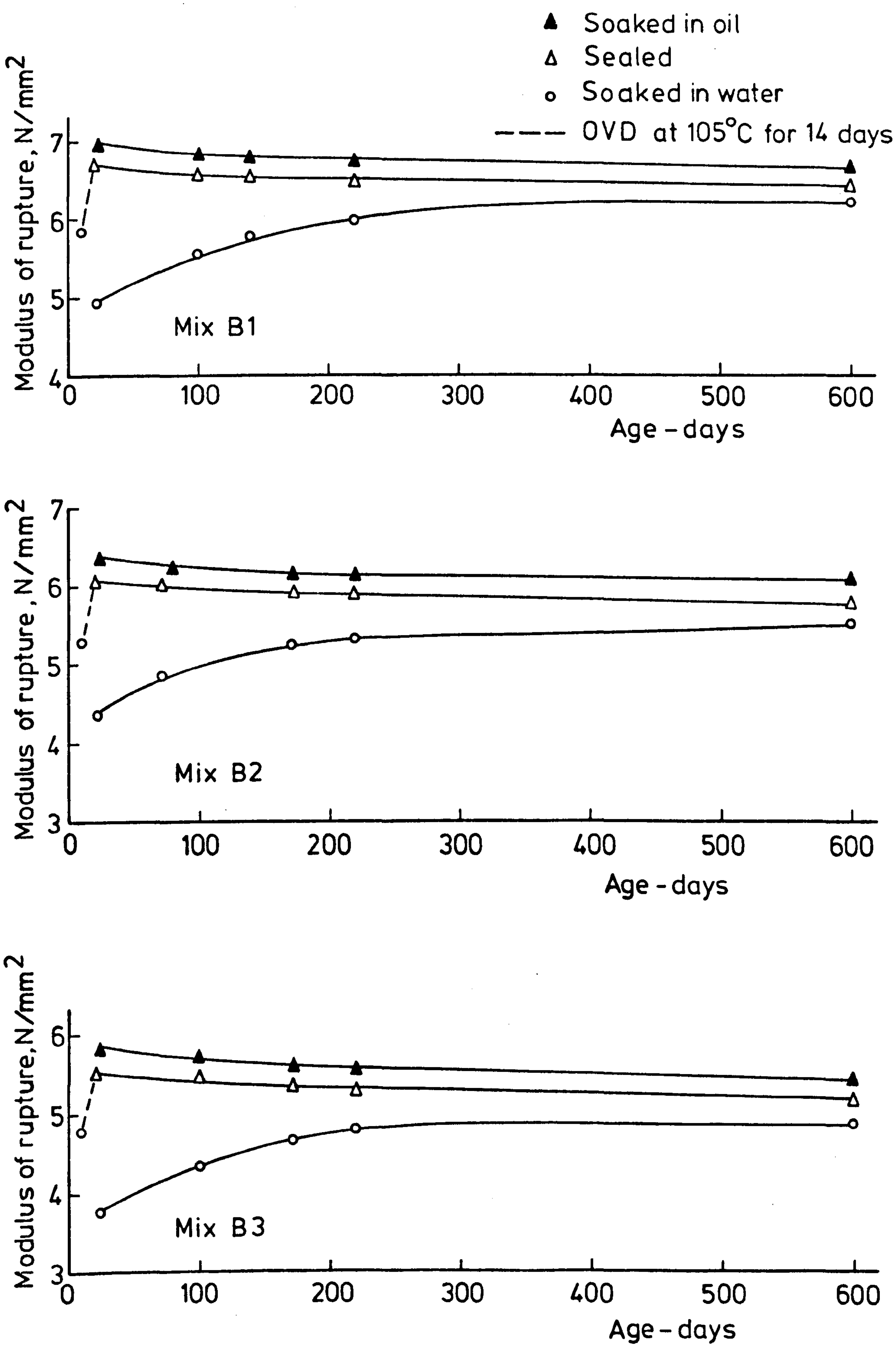


FIG. 4-23 THE EFFECT OF CURING CONDITION ON THE FLEXURAL STRENGTH OF CONCRETE AT VARIOUS AGES

reduction, the strength began to increase but it did not reach that of the dried specimen before soaking in water.

4. The modulus of rupture of the dried specimens increased significantly ($P \leqslant 0.05$) by about 3-6% for both mixes after 24 hours of soaking in oil. This increase was then followed by a continuous reduction with time due to the increase in the amount of absorbed oil. This reduction was significant ($P \leqslant 0.05$) and varied between 3-6.5% for Mix A (with an oil content variation of 5.22-5.51% by weight) and 4-7.5% for Mix B(with an oil content variation of 4.5-4.96% by weight) within a soaking period of 600 days compared with that of 1-day oil soaked strength.

4.4.5 The Effect of Oil Saturation on the Elastic Modulus of Concrete.

4.4.5.1 The Stress-Strain Curve.

The static modulus of elasticity was determined from stress-strain curves for prisms tested to failure. Typical curves are given in figs. (4.24, 4.25) for concrete of different w/c ratios and tested after various storage conditions. The following observations are made:

1. For most specimens with the same w/c ratio, the stress-strain relationship was identical at low loadings irrespective of the curing condition. For each concrete mix, figs. (4.24, 4.25), the relationships were almost identical for the sealed and oil saturated specimens to about 15% of the failure stress after which the curves started to branch out. Dry specimens soaked in water for 600 days showed greater linearity and stiffness than those saturated in oil or sealed as a reference.

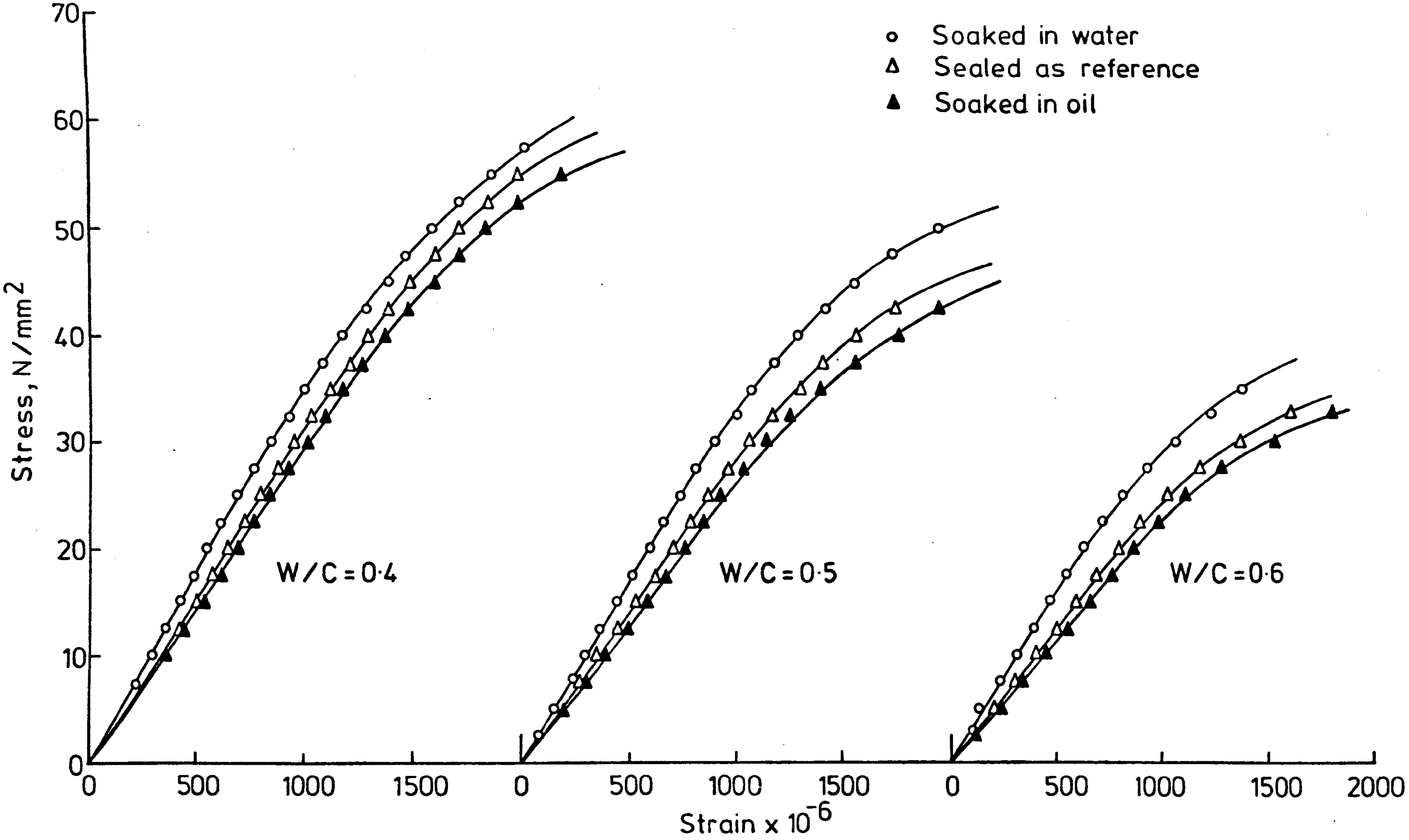


FIG. 4-24 THE EFFECT OF CURING CONDITION ON STRESS-STRAIN CURVE OF CONCRETE-MIX A (AT 600 DAYS)

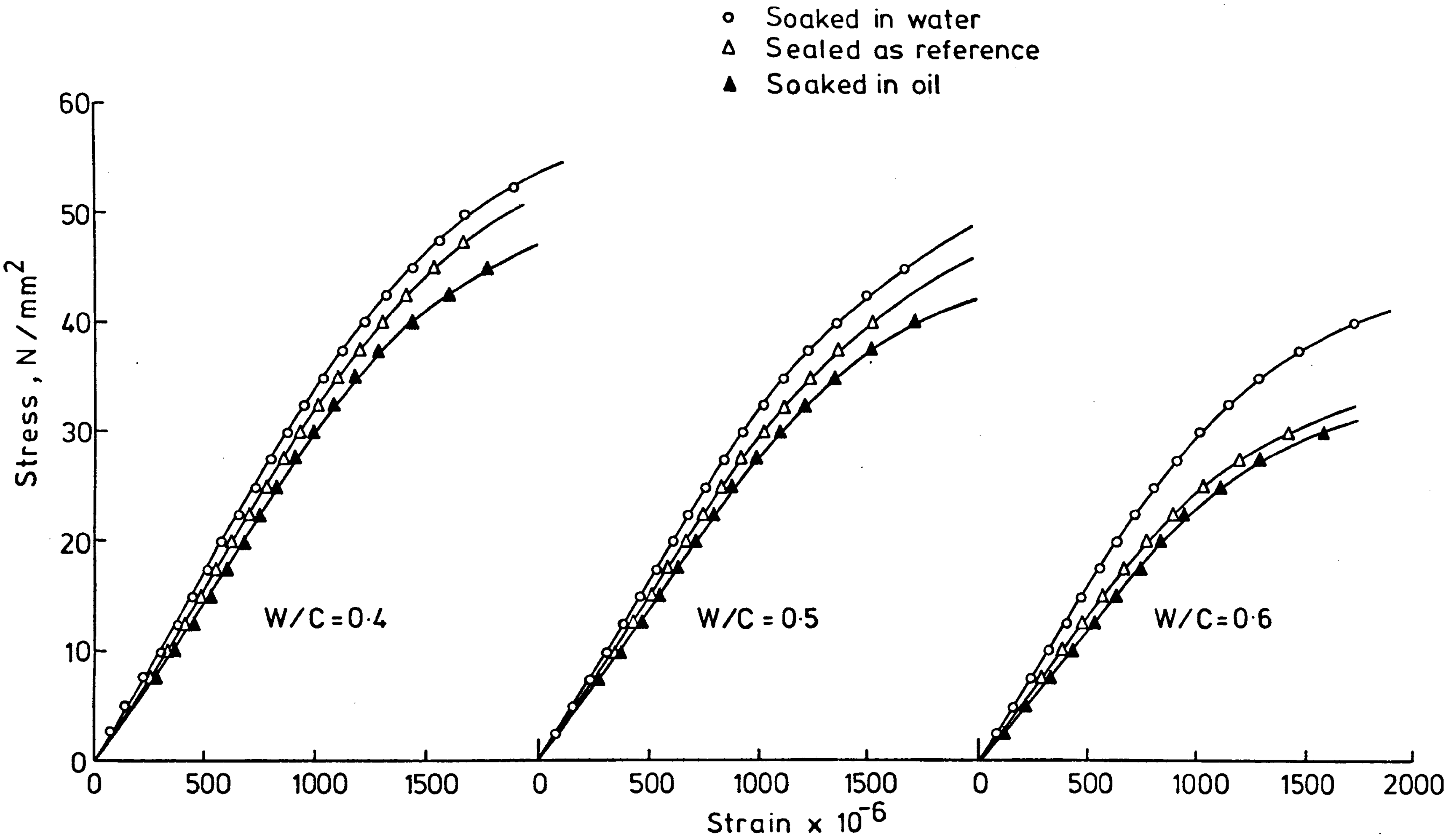


FIG. 4-25 THE EFFECT OF CURING CONDITION ON THE STRESS-STRAIN CURVE OF CONCRETE-MIX B (AT 600 DAYS)

2. The point at which the curves deviated from proportionality (crack initiation point) varied in the various test specimens between 17.5-35.0 N/mm^2 for Mix A (using 10 mm aggregate) and 15.5-32.5 N/mm^2 for Mix B (using 20 mm aggregate) depending on the w/c ratio and the curing condition. These represent between 45-55% of failure stress. For oil saturated and sealed specimens, the crack initiation point occurred at a stress level (θ_i) lower than that for the water saturated specimens by about 14-22% depending on the w/c ratio.

3. Tables (4.13,4.14) show the values of the failure stress. These values decreased as a result of crude oil saturation compared to the companion specimens of other curing conditions tested at the same age. For Mix A, the failure stress of the 600 days oil saturation prisms were between 2-4.5% higher than that of the sealed prisms and 15.5-18.5% lower than that of the water saturated specimens tested at the same age. For 20 mm aggregate concrete (Mix B), crude oil saturation for 600 days leads to between 3.5-6.5% reduction in the failure stress compared to the sealed specimens and 20-25% reduction compared to the corresponding water saturated specimens.

4.4.5.2 The Effect of Oil Saturation on the Elastic Modulus of Concrete at Various Ages.

Tables (4.13,4.15) give the values of elastic modulus of concrete for both mixes at various storage conditions, and plotted as a function of age as shown in figs. (4.26,4.27) for Mix A and B respectively. Three samples were crushed at each test and the mean value was taken. The 95% confidence limit for each mean was calculated as described in

Mix Designation	w/c Ratio	28 - day		150 - day		275 - day		400 - day	
		E KN/mm^2 (S.D.)	fc N/mm^2	E KN/mm^2 (S.D.)	fc N/mm^2	E KN/mm^2 (S.D.)	fc N/mm^2	E KN/mm^2 (S.D.)	fc N/mm^2
A1	0.40	33.95 (±1.36	58.0	35.75 (±1.20)	67.0	37.88 (±1.91)	69.5	39.00 (±0.97)	73.0
A2	0.50	29.88 (±1.06)	48.0	31.50 (±0.91)	54.0	33.50 (±1.15)	57.0	34.63 (±.70)	59.0
A3	0.60	26.13 (±0.81)	37.5	27.75 (±0.61)	43.0	29.88 (±.90)	45.0	31.03 (±.73)	48.0
B1	0.40	34.08 (±1.64)	56.0	35.88 (±1.84)	63.0	37.60 (±1.02)	65.0	38.95 (±1.39)	69.5
B2	0.50	29.28 (±1.31	45.0	31.33 (±1.17)	53.5	33.10 (±1.36)	56.2	34.30 (±1.71)	58.0
B3	0.60	25.88 (±1.44)	35.5	27.45 (±1.19)	42.5	29.35 (±1.13)	46.0	30.83 (±1.12)	47.5

*95% Confidence limits are given in brackets.

Table (4.13) Static Modulus of elasticity and failure stress of concrete prisms continuously cured in water for different ages - KN/mm^2.

Mix Designation	w/c Ratio	Sealed as Reference			Resoaked in Oil			Resoaked in Water		
		Failure stress (fc) N/mm^2	Critical stress (θ_c) N/mm^2	Initiation stress (θ_1) N/mm^2	Failure stress (fc) N/mm^2	Critical stress (θ_c) N/mm^2	Initiation stress (θ_1) N/mm^2	Failure stress (fc) N/mm^2	Critical stress (θ_c) N/mm^2	Initiation stress (θ_1) N/mm^2
A1	0.4	56.0	50.0	30.0	57.0	46.0	30.0	67.5	55.0	35.0
A2	0.5	44.2	37.5	22.5	45.0	35.5	22.5	55.0	42.5	27.5
A3	0.6	33.5	27.5	17.5	35.0	25.0	17.5	43.0	31.5	22.5
B1	0.4	53.5	40.0	27.5	51.0	35.0	27.5	64.0	45.0	32.5
B2	0.5	41.5	33.5	22.0	40.0	30.0	20.0	52.0	37.5	25.0
B3	0.6	33.0	23.0	17.0	30.8	22.0	15.5	40.9	30.0	21.0

Table (4.14) The Failure Stress, Critical Stress θ_{cr}, and the Initiation Stress θ_i for concrete specimens stored for 600 days at various storage conditions.

Mix Designation	w/c Ratio	70 - day			155 - day			325 - day			600 - day		
		Sealed (SE)	Soaked in oil (SO)	Soaked in Water (SW)	Sealed (SE)	Soaked in oil (SO)	Soaked in Water (SW)	Sealed (SE)	Soaked in oil (SO)	Soaked in Water (SW)	Sealed (SE)	Soaked in oil (SO)	Soaked in Water (SW)
A1	0.4	33.63 (±2.86)	33.13 (±2.01)	29.88 (±2.54)	32.88 (±3.58)	32.50 (±3.23)	31.88 (±3.45)	32.63 (±3.65)	31.88 (±1.94)	34.38 (±3.80)	32.48 (±2.51)	31.38 (±2.76)	37.98 (±3.31)
A2	0.5	29.25 (±2.83)	28.08 (±1.87)	26.00 (±2.06)	28.75 (±2.29)	28.25 (±2.59)	27.75 (±1.86)	28.63 (±2.24)	27.88 (±2.93)	29.88 (±2.36)	28.13 (±2.86)	26.88 (±2.66)	32.88 (±3.23)
A3	0.6	25.75 (±1.84)	25.38 (±1.29)	23.88 (±2.61)	25.38 (±2.73)	24.62 (±2.73)	25.13 (±3.35)	25.00 (±2.23)	24.13 (±3.03)	26.75 (±2.36)	24.38 (±2.26)	23.13 (±2.24)	28.88 (±2.00)
B1	0.4	33.63 (±3.31)	33.38 (±3.03)	30.43 (±2.01)	33.38 (±3.65)	32.75 (±3.11)	33.00 (±3.6)	33.00 (±3.42)	32.38 (±2.54)	34.75 (±2.76)	32.38 (±2.61)	31.13 (±1.74)	37.5 (±3.41)
B2	0.5	29.13 (±3.11)	28.63 (±3.31)	27.25 (±3.50)	28.50 (±2.93)	27.88 (±4.13)	29.63 (±4.20)	28.38 (±4.30)	27.38 (±4.05)	30.50 (±4.35)	27.63 (±3.43)	26.38 (±2.83)	32.38 (±3.78)
B3	0.6	25.13 (±3.63(	24.63 (±3.58)	23.45 (±2.12)	24.63 (±3.08)	28.88 (±2.68)	25.38 (±2.83)	24.13 (±1.76)	23.25 (±3.73)	26.13 (±2.68)	23.63 (±2.86)	22.35 (±2.83)	28.50 (±3.36)

*95% Confidence limits are shown in brackets.

Table (4.15) Static Modulus of Elasticity of Dried Concrete Specimens Stored in Different Curing Condition-Kn/mm^2.

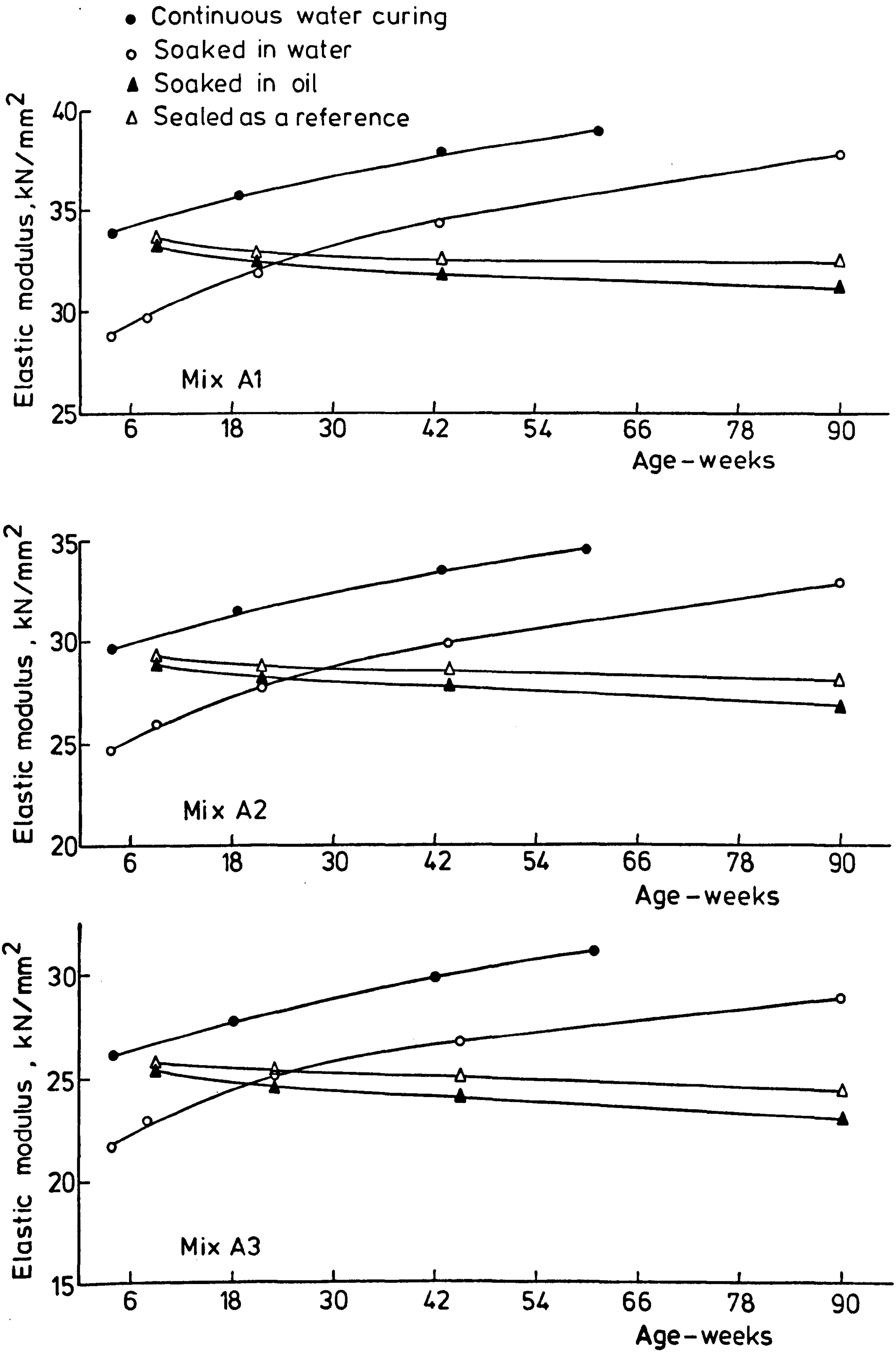


FIG. 4-26. VARIATION OF THE ELASTIC MODULUS OF CONCRETE WITH PERIOD OF STORAGE IN OIL

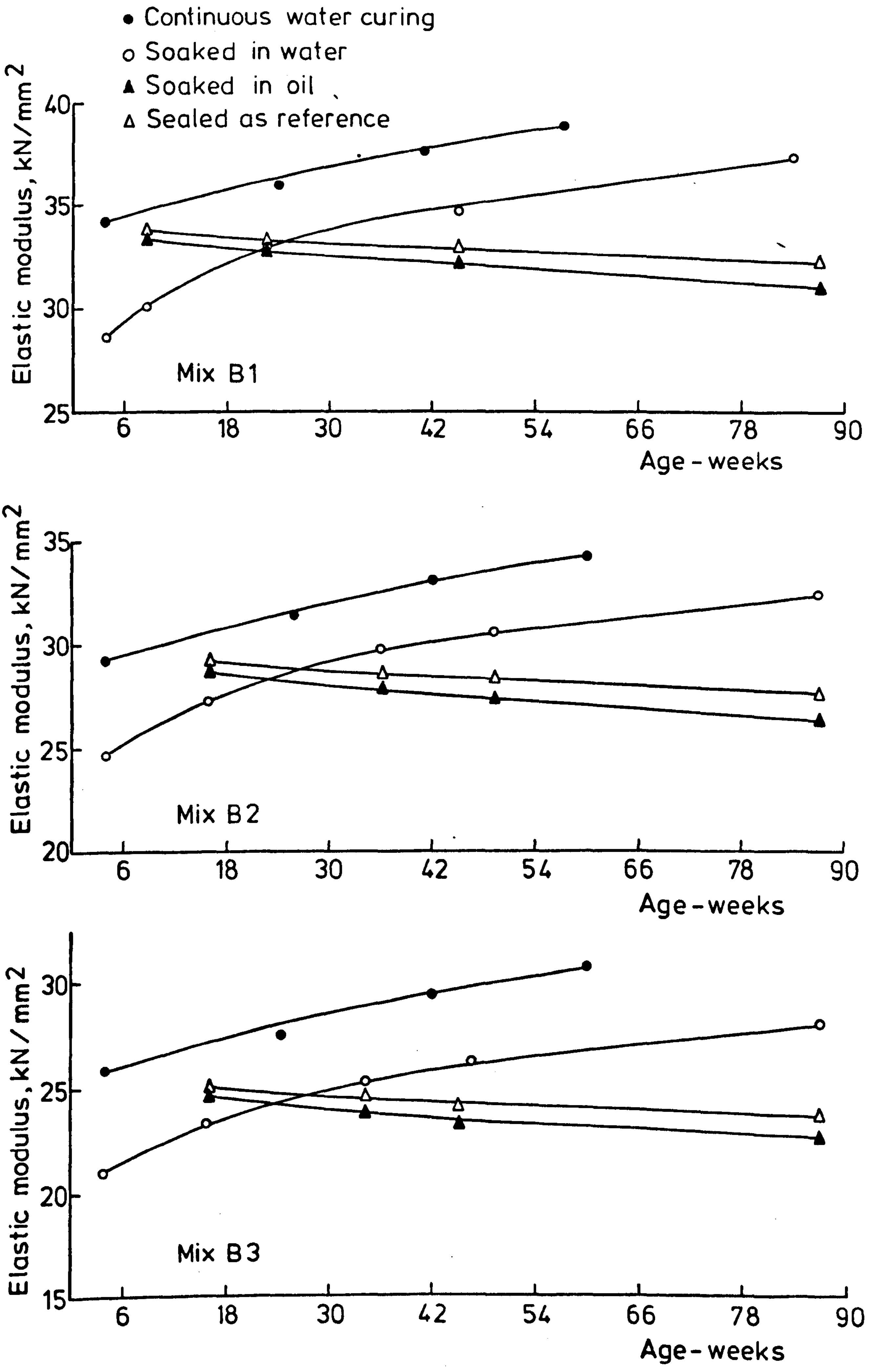


FIG. 4-27. VARIATION OF THE ELASTIC MODULUS OF CONCRETE WITH THE PERIOD OF STORAGE IN OIL

section (4.3.1) and given in brackets in tables (4.13,4.15). These values suggest the following observations:-

1. The elastic modulus of a continuous moist cured concrete was gradually increased with time. The variation in this increase was between 14-19% within a soaking period of 400 days compared to that of the 28-day moist cured. This increase was independent of maximum size of aggregate used in the mix. The relationship between the elastic modulus of the continuously moist cured specimen and the curing period for the test results shown in table (4.13) can be expressed by the following formula:

$$\frac{E_t}{E_{28}} = [0.92 - 0.2\ w/c]\ t^{0.12\ w/c} \quad ..... \quad (4.7)$$

where

E_t = modulus of elasticity at any time during moist curing - KN/mm^2.

E_{28} = modulus of elasticity of the 28-day water curing - KN/mm^2.

t = water curing period - days.

w/c = water-cement ratio.

The average coefficient of correlation was 97.76, and the maximum error was approximately equal to ± 3% compared with the results obtained by experiment.

2. The modulus of elasticity of oven dry concrete specimens was reduced by between 16-17.5% for 10 mm aggregate concrete (Mix A) and 18-20% for 20 mm aggregate concrete (Mix B) after 24 hours of soaking in water. This was then followed by a gradual increase with time, but the modulus of elasticity did not reach that of the continuously wet cured concrete. After

600 days of moist curing, the increase in the elastic modulus was between 10-11% higher than that at 28-days moist curing.

3. The elastic modulus of the sealed concrete specimens showed an insignificant ($P \leqslant 0.05$) reduction with time. This reduction was between 3.5-6% within a storing period of 600 days.

4. The dried concrete specimens soaked in oil showed a continuous reduction in the modulus of elasticity as the storage period increased, i.e. the amount of absorbed oil increased. This reduction varied between 6-10% for Mix A (with an oil content variation of 5.1-5.72% by weight) and 7.5-11% for Mix B (with an oil content variation of 4.51-5.02% by weight) within a soaking period of 600 days compared with that of the dry concrete before soaking in oil. This reduction was significant ($P \leqslant 0.05$) for concrete of Mix A using w/c of 0.6 and for concrete of Mix B independent of w/c ratio.

5. The relationship between the modulus of elasticity at any period and that of the 28-days water cured, for each curing procedure, for the test results presented in figs. (4.26,4.27) can be expressed by the following formula:-

$$\frac{E_t}{E_{28}} = a\ t^b \qquad \ldots\ldots \qquad (4.8)$$

The constants a and b are given in table (4.16).

Concrete Mix	Storage Condition	a	b	r
A	SE	1.03	-0.0145	98.74
	SO	1.06	-0.025	98.88
	SW	0.80	0.040	94.20
B	SE	1.06	-0.021	97.72
	SO	1.09	-0.031	97.59
	SW	0.80	0.039	95.69

Table 4.16) Constants of regression - equation (4.8) modulus of elasticity vs. soaking period.

4.4.5.3 The Effect of Oil Saturation on Elastic Modulus of Concrete at Various w/c Ratios.

The variation of elastic modulus with w/c ratio is illustrated in fig. (4.28) for oven dried specimens soaked in water, sealed as a reference and soaked in oil for 600 days. The following observations are made:

1. The dry concrete soaked in water possesses a higher elastic modulus than sealed concrete which had a higher modulus than oil saturated concrete. The elastic modulus of oil saturated concrete was between 14-15.5% less than for water saturated samples after 600 days of oil soaking.

2. Fig. (4.28) shows that the elastic modulus decreased with an increase in the w/c ratio following an exponential relationship of the form:-

$$E = E_o e^{b(w/c)} \quad ..... \quad (4.9)$$

The constants E_o and b were obtained by regression analysis and are given in table (4.17).

Mix Design	Storage condition	E_o	b	r
Mix A	600 days SW	65.0	-1.356	99.88
	600 days SE	57.6	-1.434	99.92
	600 days SO	57.7	-1.525	99.92
Mix B	600 days SW	67.0	-1.460	99.89
	600 days SE	60.7	-1.575	99.93
	600 days SO	60.4	-1.656	99.90

Table (4.17) Constants of regression - Equation (4.9) Elastic modulus Vs. w/c ratio.

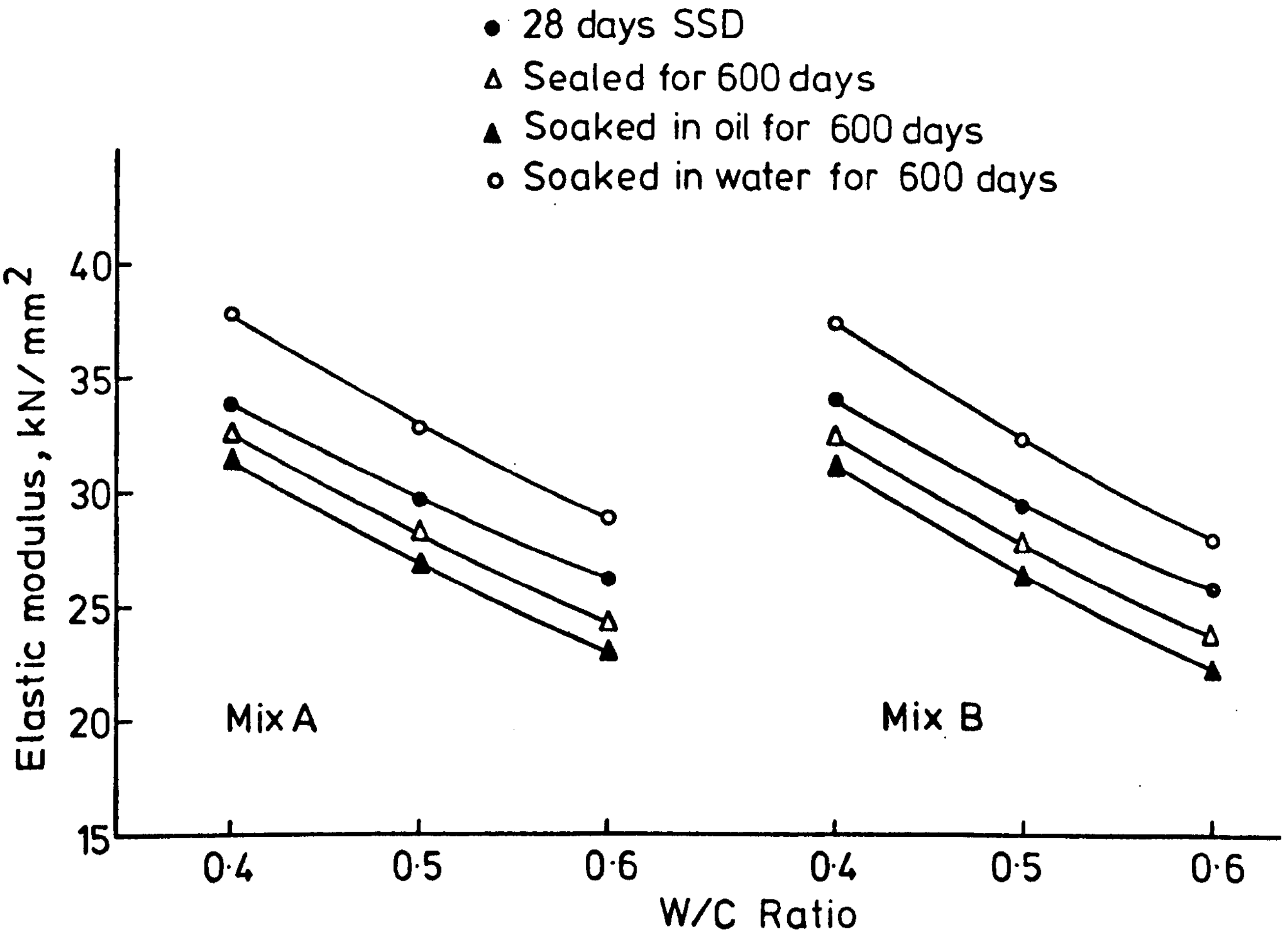


FIG. 4-28. EFFECT OF STORAGE CONDITIONS AND W/C RATIO ON THE ELASTIC MODULUS OF CONCRETE

4.4.6 Volumetric Changes under Compressive load.

It is well known (133) that when concrete is subjected to increasing compressive load, the volume decreases until at a certain stress (θ_c), the critical stress, the volume starts to increase. In the present investigation, the volumetric strain was calculated from the strain relationship(133):

$$\frac{\Delta V}{V} = \varepsilon_L - 2\varepsilon_T \quad \ldots\ldots \quad (4.10)$$

where ε_L is the longitudinal strain and ε_T the lateral strain.

The changes in volumetric strain with stress are illustrated in figs. (4.29, 4.30) for both Mix A and B respectively. The following observations are made:-

1. At low loads, the oil saturated concrete decreased in volume more than the water saturated or sealed specimens. But as the load continued beyond the critical stress (θ_c), the gross volume of the oil saturated concrete increased at a high rate compared to the other specimens. The dilation of water saturated and sealed prisms was less than for oil saturated prisms for all concrete mixes.

2. The critical stress, θ_c, denoting the points at which the curves deviated from linearity, due to unstable crack propagation(134) was highest for the dried specimens soaked in water for 600 days and least for oil saturated specimens soaked for the same period. The critical stress (θ_c), given in table (4.14), ranges between 0.71-0.85 for Mix A and 0.63-0.75 for Mix B as a percentage of the ultimate stress.

4.4.7 Poisson's Ratio of Oil Saturated Concrete.

Changes in Poisson's ratio with stress for oil saturated concrete compared to that of the sealed and water saturated

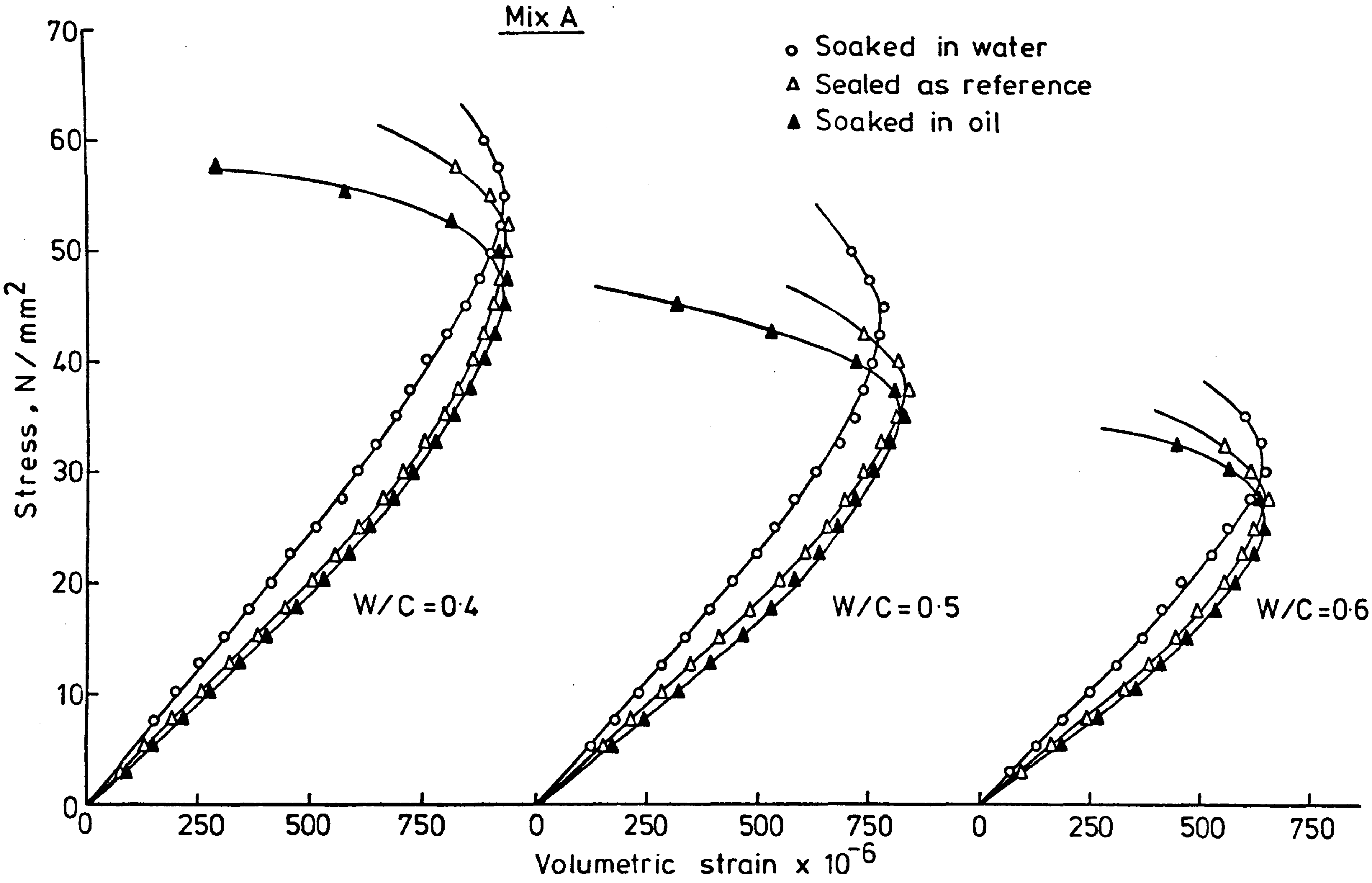


FIG. 4-29. TYPICAL PLOTS OF VOLUMETRIC CHANGES UNDER COMPRESSIVE LOAD (AT 600 DAYS)

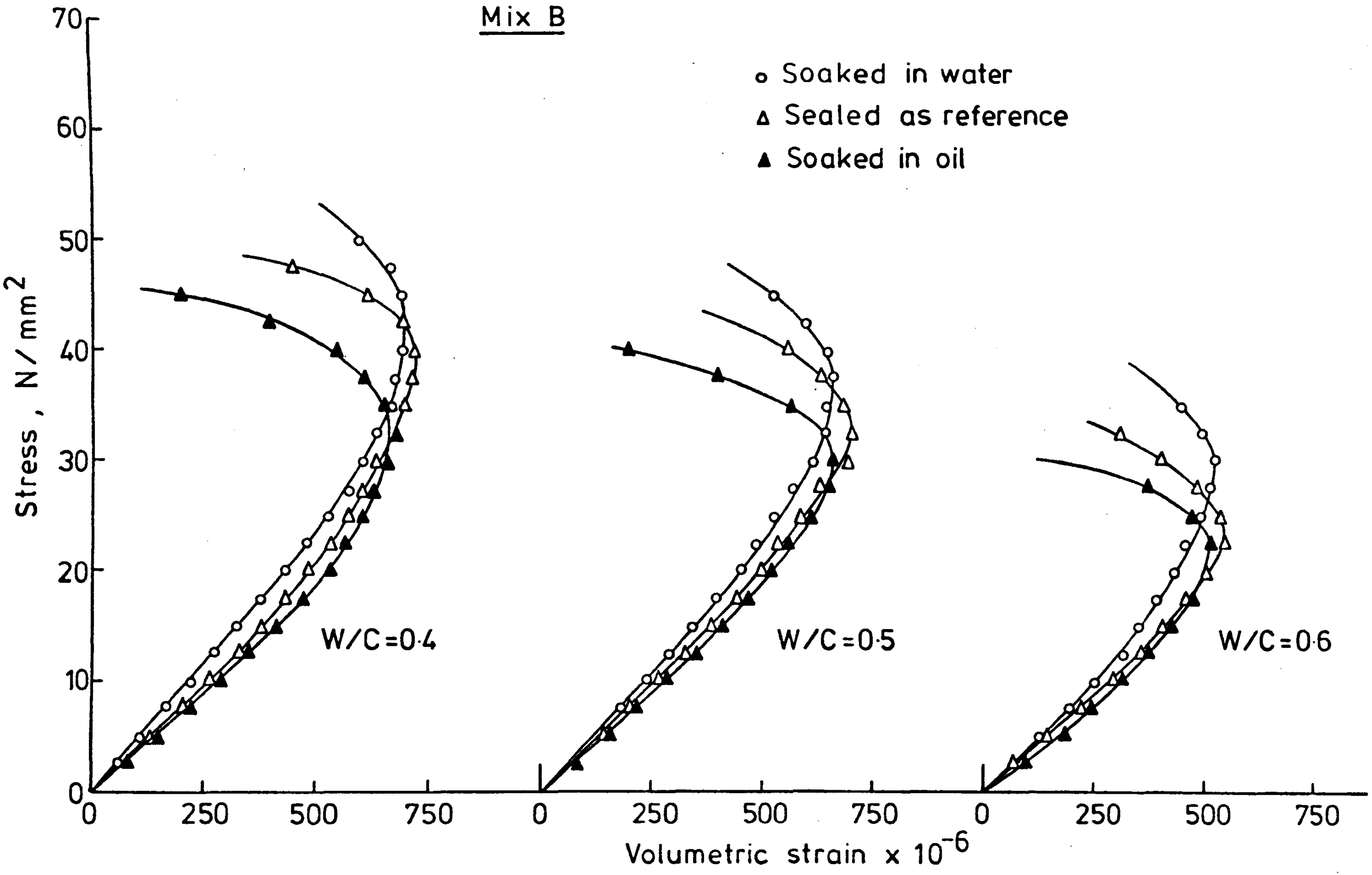


FIG. 4-30. TYPICAL PLOTS OF VOLUMETRIC CHANGES UNDER COMPRESSIVE LOAD (AT 600 DAYS)

prisms are illustrated in figs. (4.31, 4.32). The following observations are made:

1. The Poisson's ratio is between 0.1 and 0.2 at low stress, i.e. between 20-40% of ultimate stress, but the value increases rapidly as the stress increased.

2. The Poisson's ratio of the oil saturated specimens were generally higher than those for sealed or water saturated specimens of the same w/c ratio. The water saturated had the smallest values. The increase in Poisson's ratio at high loads was greater for oil saturated prisms.

4.4.8 The Ultrasonic-Pulse Velocity of Concrete Cubes under Uniaxial Compressive Load.

In the present investigation, the pulse velocity technique was used to detect the onset of cracking in a cube compression test specimen. Cubes soaked in oil for 600 days were compared to others stored for the same period in water or sealed in polythene bags as a reference. The measured ultrasonic pulse velocity for all curing conditions is plotted against applied stress as shown in figs. (4.33, 4.34), for Mixes A and B respectively. The following observations are made:

1. The ultrasonic pulse-velocity for the concrete specimens soaked in water was higher than those soaked in oil or kept sealed as a reference. The reference specimens generally had the smallest values. Water saturated cubes were between 2.5-5% higher than those for oil saturated specimens while reference cubes were between 5-7.5% less.

2. The pulse velocity remains constant up to a certain stress (θ_i) less than the ultimate stress and then decreased sharply, thus marking the onset of cracking across the path of the

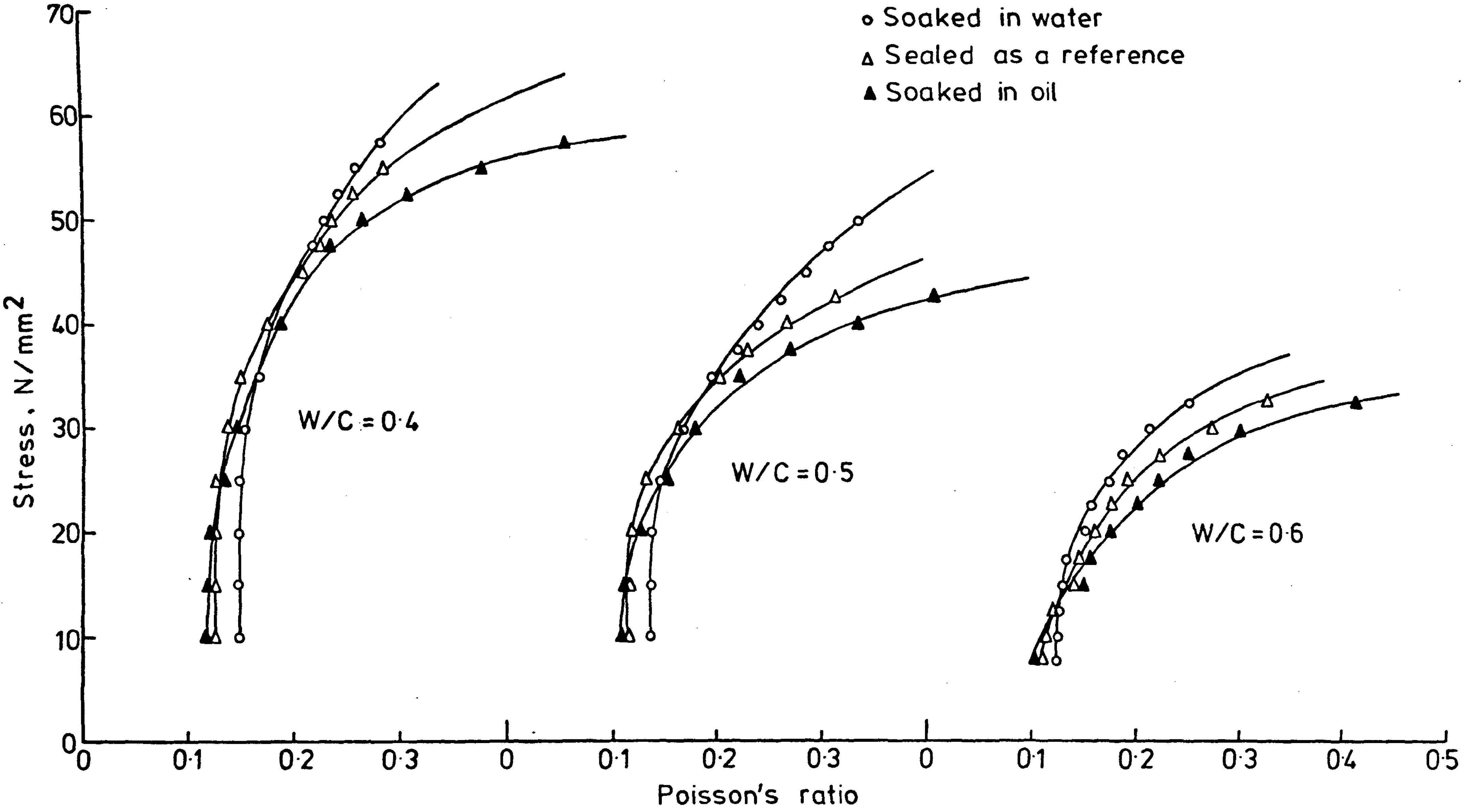


FIG. 4-31. TYPICAL PLOTS OF POISSON'S RATIO CHANGES OF CONCRETE (MIX A) UNDER COMPRESSIVE LOAD (AT 600 DAYS)

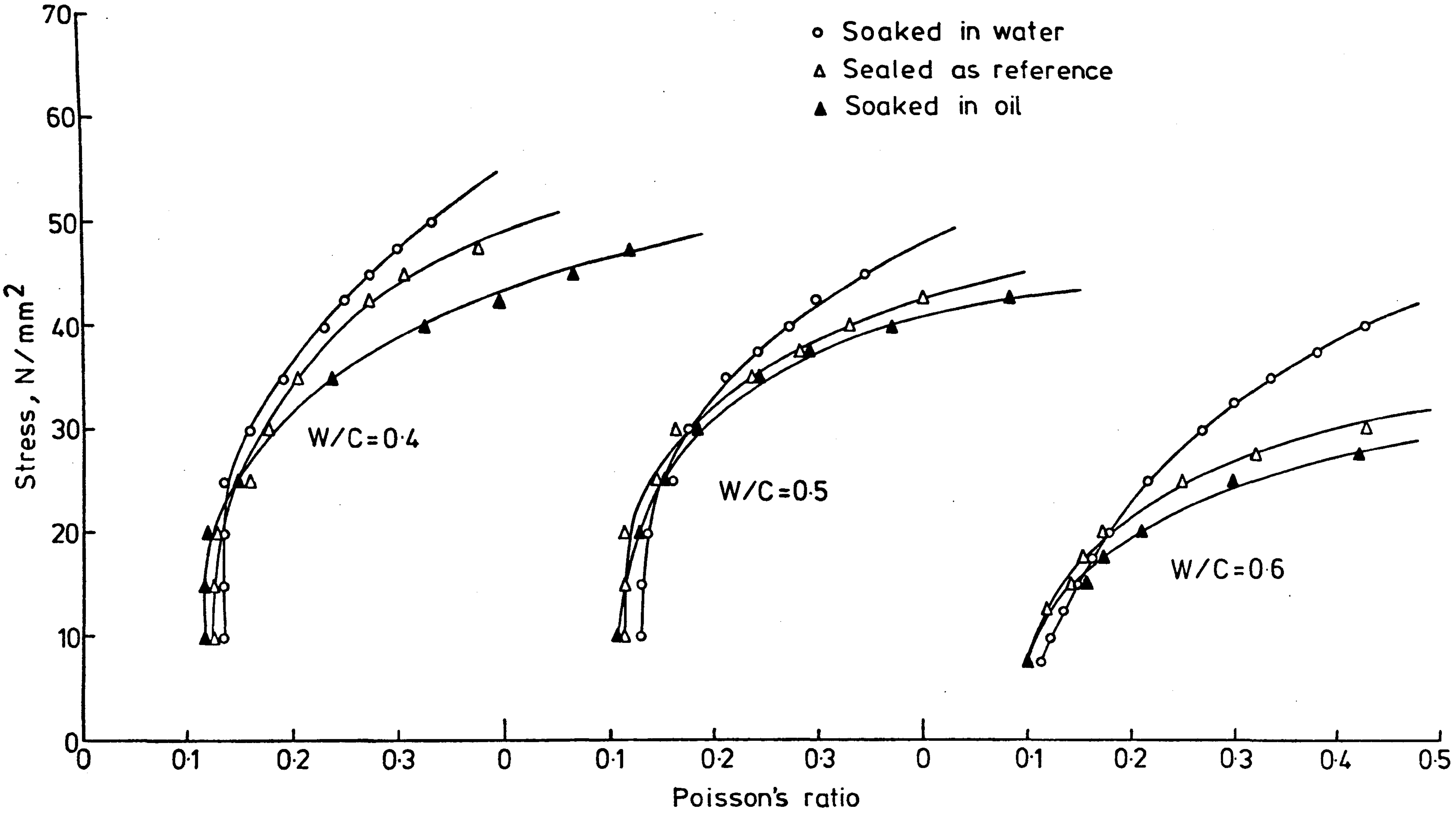


FIG. 4-32. TYPICAL PLOTS OF POISSON'S RATIO CHANGES OF CONCRETE (MIX B) UNDER COMPRESSIVE LOAD (AT 600 DAYS)

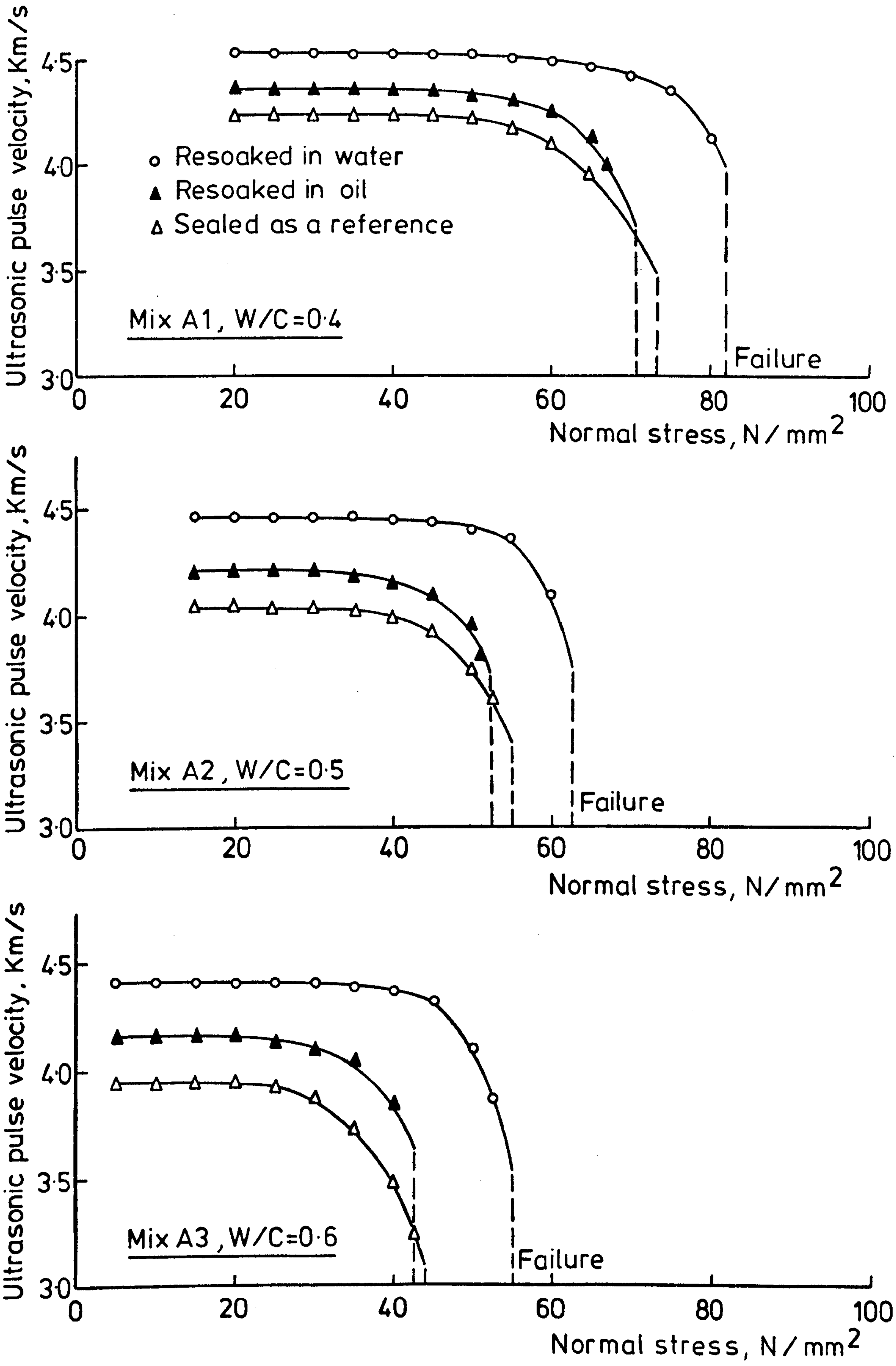


FIG. 4-33. THE VARIATION OF PULSE VELOCITY WITH APPLIED NORMAL STRESS OF CONCRETE CUBE SOAKED AT DIFFERENT CURING CONDITION FOR 600 DAYS

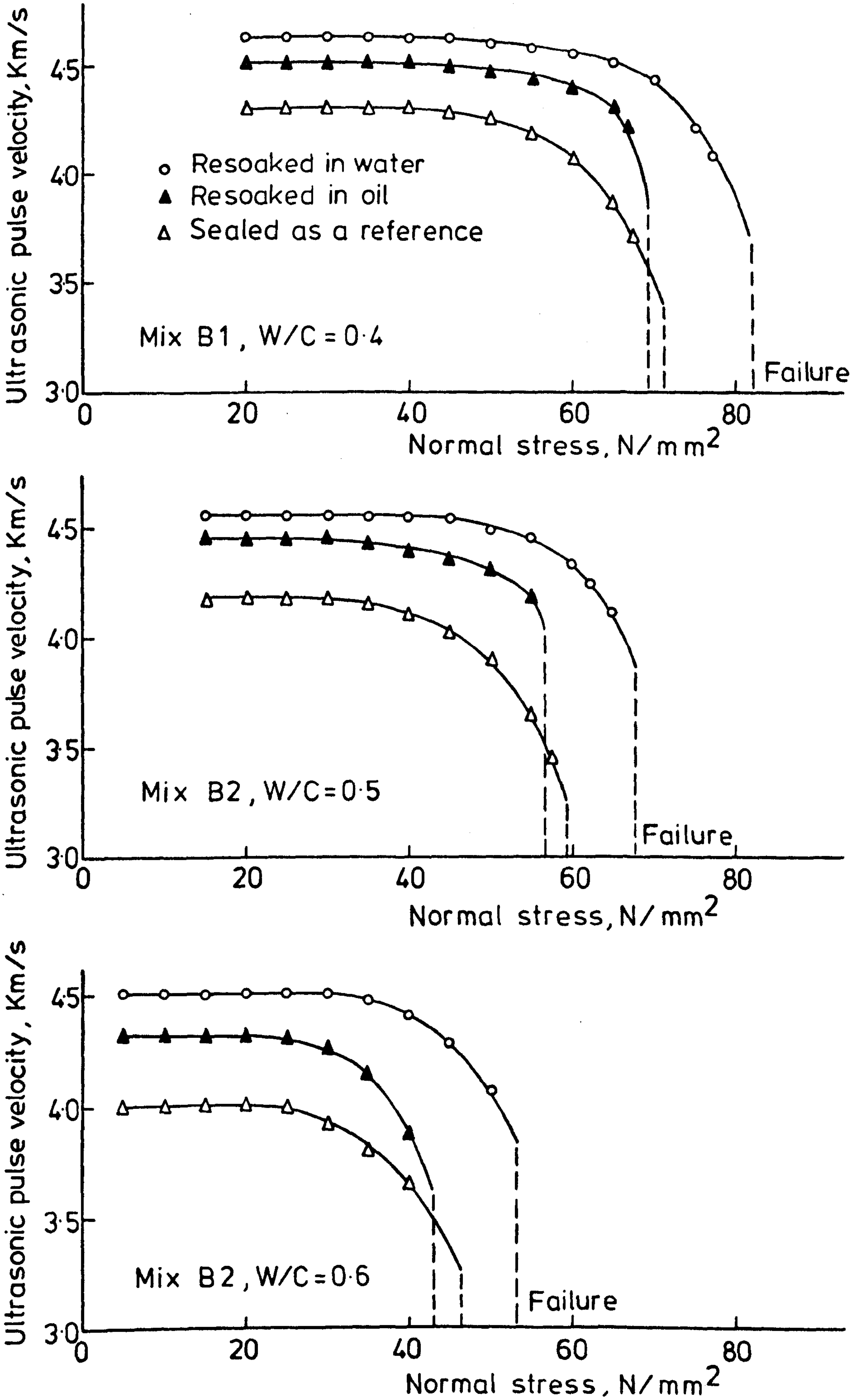


FIG. 4-34 THE VARIATION OF PULSE VELOCITY WITH APPLIED NORMAL STRESS OF CONCRETE CUBE SOAKED AT DIFFERENT CURING CONDITION FOR 600 DAYS

pulse at these particular loads.

3. The load at which the initial cracking occurred depends upon the strength of the concrete and the uniformity of the stress distribution. These loads divided by the corresponding failure load are given in table (4.18) as a percentage for the different concrete. The load at which the first crack occurred during the compression test was generally higher for those specimens soaked in water than those for oil saturated and reference specimens.

Mix	Sealed (SE)			Soaked in oil (SO)			Soaked in water (SW)		
	P.V. Km/s	θ_i N/mm^2	% of Failure Stress	P.V. Km/s	θ_i N/mm^2	% of Failure Stress	P.V. Km/s	θ_i N/mm^2	% of Failure Stress
A1	4.23	44.5	61	4.36	42.0	60	4.54	52.0	63
A2	4.04	35.5	61	4.24	32.5	57	4.46	41.5	62
A3	3.95	23.5	52	4.14	22.0	51	4.40	34.5	62
B1	4.30	42.0	59	4.53	40.0	58	4.64	48.5	60
B2	4.19	32.0	56	4.46	30.0	55	4.56	40.0	60
B3	4.00	21.5	49	4.33	20.0	49	4.51	30.0	55

Table (4.18) The Pulse-Velocity (P.V.) and Initiation Stress (θ_i) for all Mixes Tested under Uniaxial Test (at 600 days).

4.4.9 Moisture Movement and Volume Changes.

The shrinkage of oil soaked concrete prisms was studied and the results were compared with those of similar but soaked in water or sealed as a reference. The variables examined were as follows:-

a. Aggregate grading: Two types of aggregate grading were used, one with 10 mm and the other with 20 mm maximum size of aggregate.

b. Mix proportions: For each aggregate grading, three concrete mixes were used as shown in table (3.1).

c. Curing condition: Three types of curing were used as described in section (3.6).

d. Age: Readings were recorded at the following ages after being dried, 1, 3, 7, 14, 25, 45, 100, 145, 200, 360 and 550 days.

The data gathered from the volume changes of concrete soaked in crude oil, soaked in water, and sealed as a reference are summarized and plotted against age in figs. (4.35) to (4.38). The results obtained showed the following:-

1. Initially dried concrete specimens showed a rather peculiar volume change when soaked in oil. They shrank slightly for a short period until reaching a maximum value, between 100-180x10^{-6} m/m, at about 25 days for both mixes and for all w/c ratios. Thereafter, the oil soaked concrete specimens began to swell with time at a very slow rate as shown in figs. (4.35) to (4.38). Specimens of 0.4 w/c ratio showed different behaviour. They swelled very little during a soaking period of about 100 days and then continued to shrink at a slower rate with time. After a soaking period of 560 days, the total shrinkage strain of oil soaked specimens varied between 50-100x10^{-6} m/m for concrete of Mix A and 100-150x10^{6} m/m for concrete of Mix B.

2. The oven dried specimens sealed as a reference shrank slightly at early age by various amounts depending on the aggregate/cement ratio and the size of the aggregate used in the mix. After a storage period of 25 days, the shrinkage

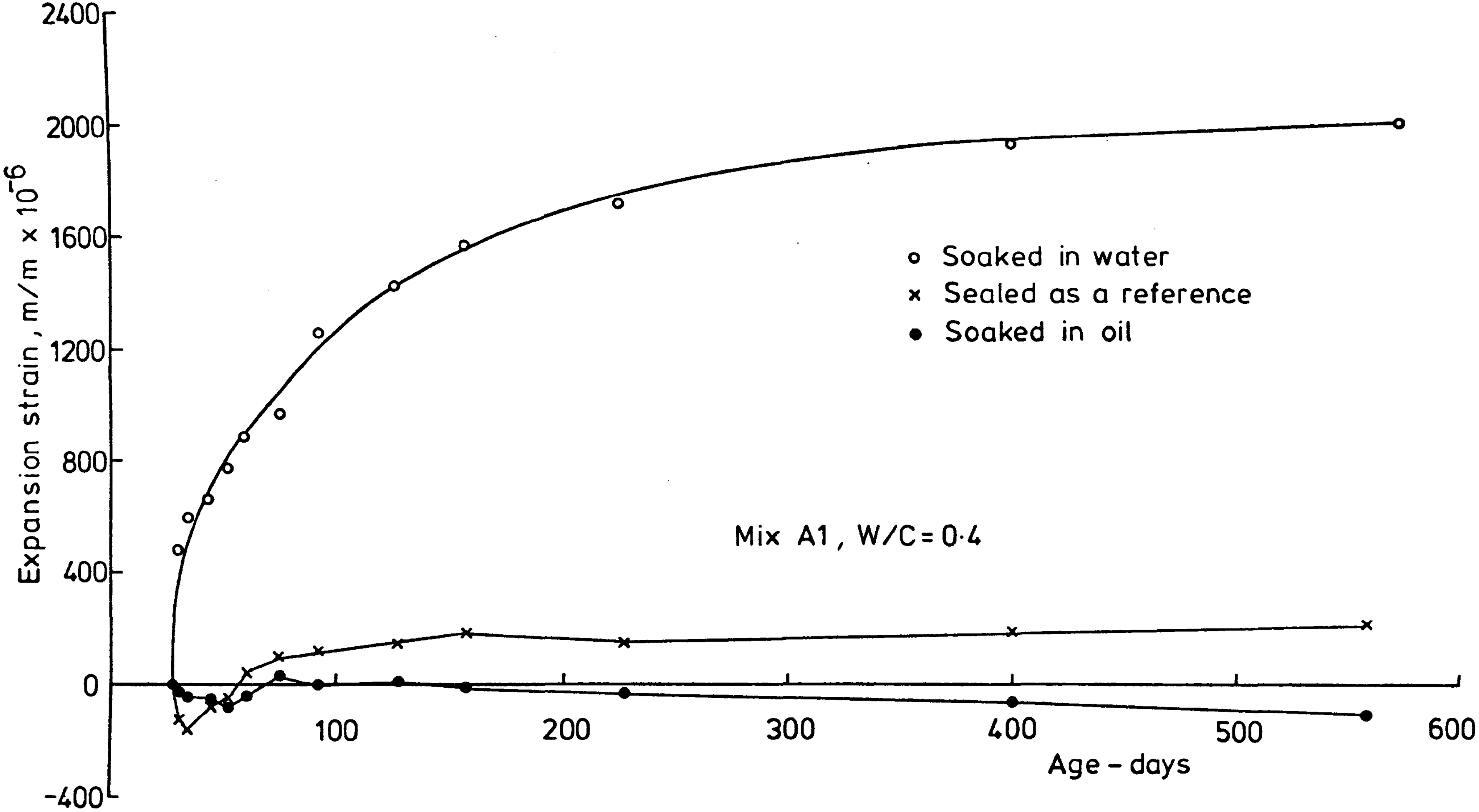


FIG. 4-35. VOLUME CHANGE OF CONCRETE Vs AGE

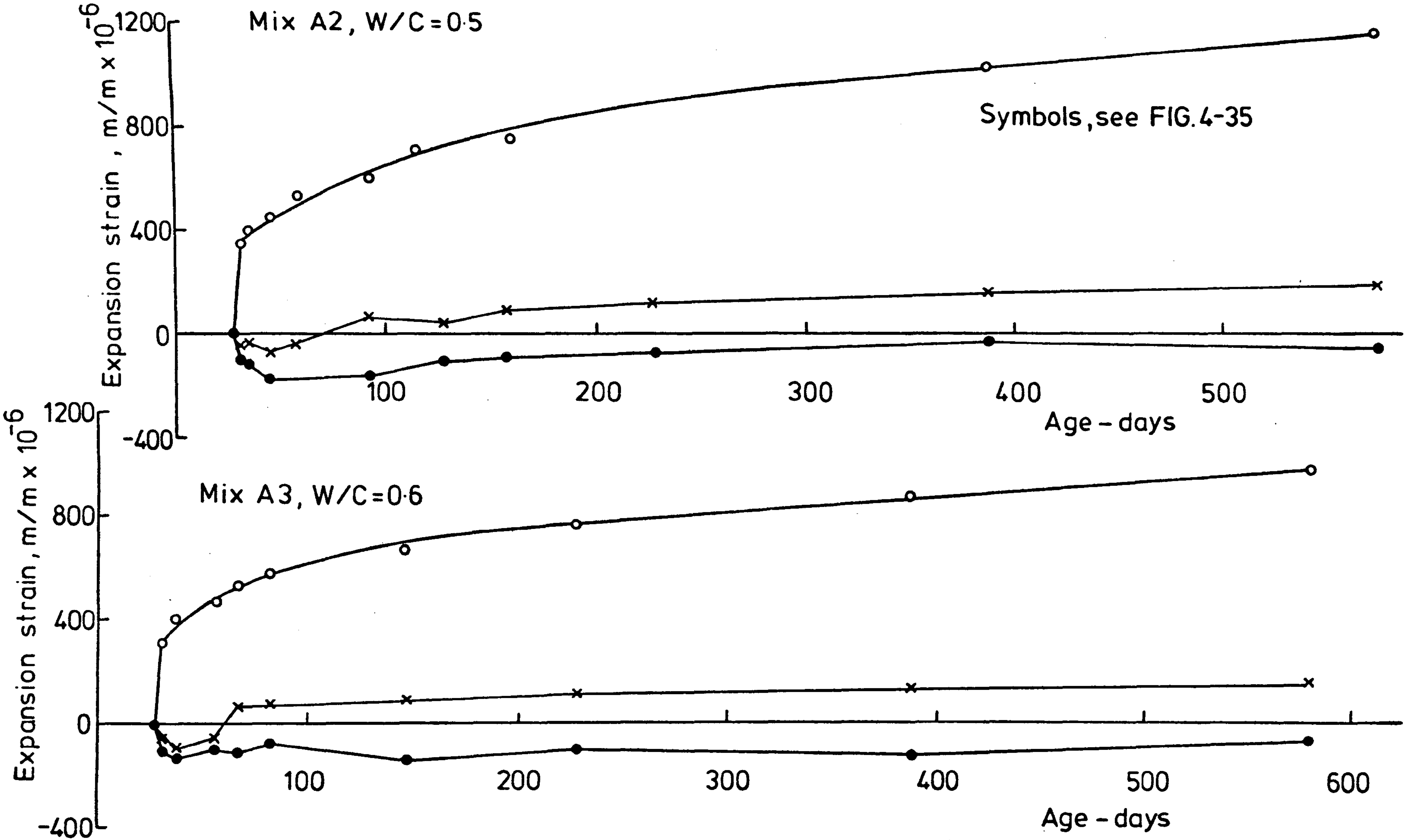


FIG. 4-36. VOLUME CHANGE OF CONCRETE Vs AGE

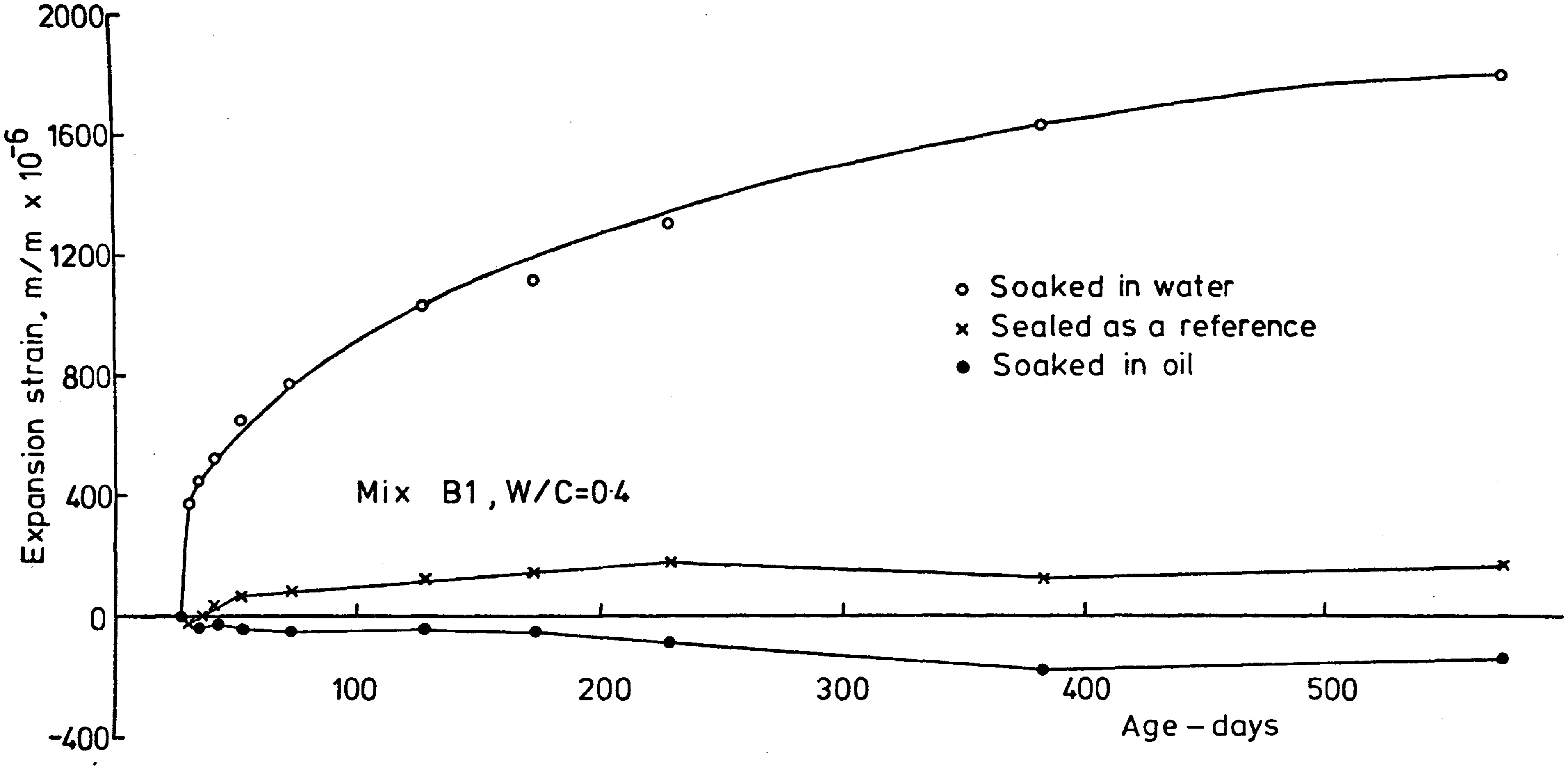


FIG. 4-37. VOLUME CHANGE OF CONCRETE Vs AGE

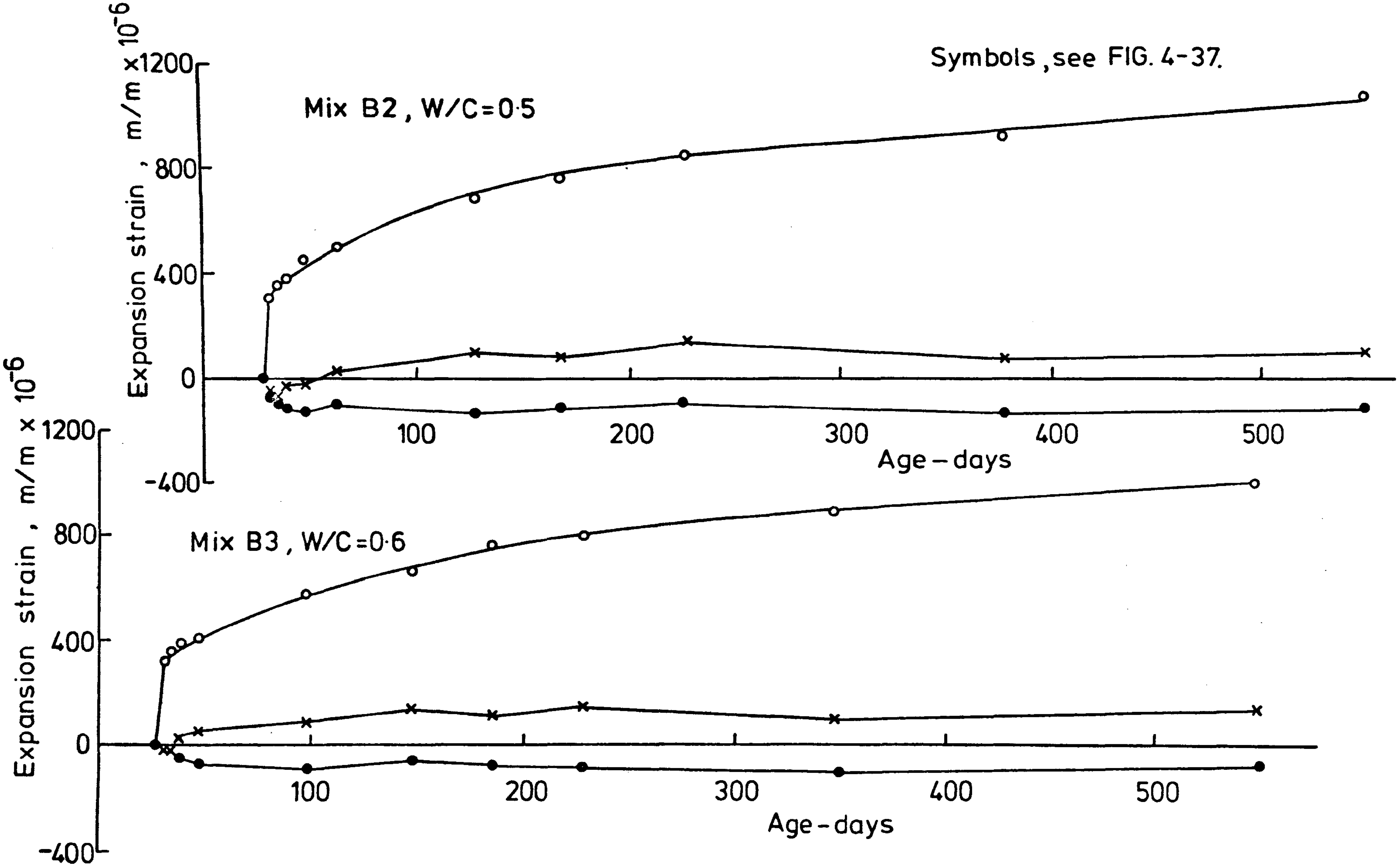


FIG. 4-38. VOLUME CHANGE OF CONCRETE Vs AGE

strain varied between 100-150x10^{-6} m/m for Mix A and 25-100x10^{-6} m/m for Mix B. Thereafter, they began to expand at a slower rate. The total expansion within a storage period of 560 days was between 150-200x10^{-6} m/m for concrete of Mix A and between 100-150x10^{-6} m/m for concrete of Mix B.

3. Dried concrete specimens soaked in water swelled immediately by various amounts depending on the mix proportion. After 50 days of soaking period, the concrete specimens swelled by about 50% of the total expansion. The maximum expansion strain varied between 1000-2000x10^{-6} m/m for concrete of Mix A and 1000-1800x10^{-6} m/m for concrete of Mix B within a soaking period of 560 days. The expansion strain was higher for the concrete mix of higher cement content and lower aggregate/cement ratio.

4.4.10 The Effect of Total Porosity on the Compressive Strength of Oil Saturated Concrete.

The porosity (cc/cc or sometimes in cc/gm) is defined as the fraction of the total volume which is occupied by pores. It plays a very important role in determining the physical and engineering properties of Portland cement pastes and consequently of concrete. The total porosity of HCP's, mortar,and concrete was calculated to study its effect on the measured properties when soaked in oil compared to other curing conditions.

4.4.10.1 Determination of Total Porosity.

The total porosity of each mortar mix (P_m) or concrete mix (P_c) used throughout this investigation was calculated as the sum of the separate porosities of cement paste and aggregate plus air content, and expressed as a percentage volume of concrete or mortar. Fegerlund (135,136) gave a generalized formula for calculating concrete porosity (P_c). This equation was rewritten as:

$$P_c = P_p . V_p + P_A (1 - V_p) + A_o \quad ..... \quad (4.11)$$

where

P_p = cement paste porosity.

V_p = volume fraction of cement paste.

P_A = aggregate porosity.

A_o = Air content.

The hardened cement paste porosity (P_p) shown in table (4.19) was calculated using the following formula(21) :-

$$P = \frac{W_e}{v} \times 100 \% \quad ..... \quad (4.12)$$

where

$W_e = W_a - W_d$

and $v = W_a - W_w$

W_e = wt. of evaporated water in sample.

W_a = wt. of sample in a saturated surface dry condition in air.

W_d = wt. of sample after drying in the oven at 105° C to constant weight.

W_w = wt. of sample while suspended in distilled water.

Equation (4.12) above assumes that the amount of evaporable water at 105° C measures the porosity of HCP, and this is one of the basic features of Powers model(28,29).

To calculate V_p, the density for each hardened cement paste was measured. Aggregate porosities, P_A, were determined by the absorption method according to BS.812, 1967. These were 1.12% for 10 mm coarse aggregate, 0.72% for 20 mm coarse aggregate, and 1.42% for fine aggregate. The air content, A_o, was determined by comparing the density of a concrete sample after 20-24 hours, d_a, with the density calculated from mix proportions, d, i.e., air free concrete. Then, A_o is given by$^{(124)}$:

$$A_o = 1 - \frac{d_a}{d} \quad \ldots\ldots \quad (4.13)$$

The calculated results are given in tables (4.20) and (4.21) for mortars and concrete respectively.

w/c Ratio	Compressive Strength 7-days SSD-N/mm^2	Wet Density SSD (gm/cm^3)	Dry Density at 105^o C (gm/cm^3)	Total Porosity (W_e) % Volume
0.30	94.0	2.29	1.98	31.52
0.35	78.0	2.22	1.87	35.52
0.40	58.0	2.15	1.77	38.10
0.50	40.0	2.02	1.58	44.85
0.60	31.0	1.95	1.45	52.16

Table (4.19) Properties of Hardened Cement Paste used in this investigation.

4.4.10.2 Variation of the Compressive Strength of Concrete with the Total Porosity.

Porosity is an important factor affecting the strength of concrete. The exponential relationship is used here to relate the strength of crude oil saturated concrete cubes to the total porosity. The strength values are given in table (4.8) and the porosities values in table (4.21). The relationship between the compressive strength and the total porosity of concrete is illustrated in fig. (4.39) and can be expressed as:

$$fc = a\, e^{-bP} \quad \ldots\ldots \quad (4.14)$$

The values of a and b are obtained by regression analysis and are given in table (4.22) for concrete stored at different conditions.

Mix Designation	w/c Ratio	HCP Density (gm/c.c.)	Volume fraction of the Mortar			Total Porosity (% Value)
			HCP porosity	Aggregate porosity	Air Content	
1:2 Mortar	0.4	2.15	0.1682	0.0073	0.034	20.63
	0.5	2.02	0.2117	0.0068	0.018	23.33
	0.6	1.95	0.2558	0.0064	0.000	25.91
1:3 Mortar	0.5	2.02	0.1706	0.0082	0.047	22.27
	0.6	1.95	0.2086	0.0078	0.030	24.33
	0.7	1.86	0.2541	0.0075	0.006	26.44

Table (4.20) Typical calculation of mortar porosity.

Mix Designation	w/c Ratio	HCP Density (gm/c.c.)	Volume fraction of the Concrete			Total Porosity (% Value)
			HCP porosity	Aggregate porosity	Air Content	
A1	0.4	2.15	0.1399	0.0072	0.0186	16.32
A2	0.5	2.02	0.1503	0.0078	0.0160	17.15
A3	0.6	1.95	0.1615	0.0081	0.0135	18.07
B1	0.4	2.15	0.1281	0.0059	0.0140	14.60
B2	0.5	2.02	0.1383	0.0064	0.0120	15.47
B3	0.6	1.95	0.1487	0.0067	0.0098	16.38

Table (4.21) Typical calculation of concrete porosity.

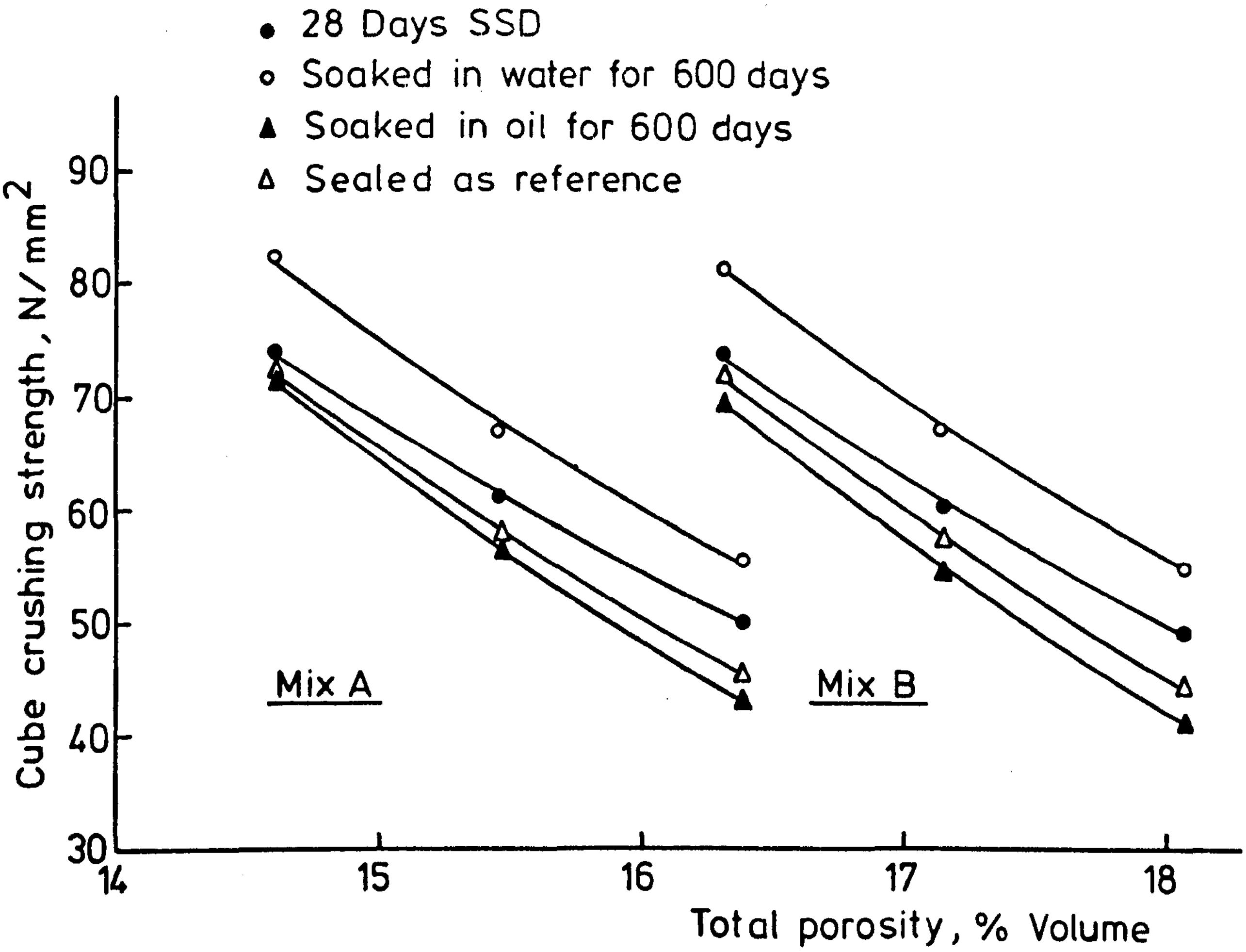


FIG. 4-39. RELATIONSHIP BETWEEN COMPRESSIVE STRENGTH AND TOTAL POROSITY OF CONCRETE

The slope, b, of the different plots describes the variation in strength with porosity. The lower the slope, the less sensitive is the strength of the concrete to the porosity. The slope of the oil saturated concrete is greater than either of the water saturated or sealed concrete. This means that the strength of oil saturated concrete is more sensitive to the porosity, in other words, the presence of oil in the pores reduces the strength compared to the presence of water or for relatively empty pores of the sealed specimens.

4.4.11 Oil Absorption Characteristics of HCP's and Mortars.

4.4.11.1 Introduction.

Different mix proportions and w/c ratios were used to study the effect of different porosity on the amount of absorbed oil and its effect on the measured properties. In the present investigation, a gravimetric method was used to measure the amount of absorbed oil. This method is simple and depends on the accuracy of the weighing facilities available. All the specimens were weighed after being dried in an oven at 105° C for a sufficient period to reach a constant weight and after being soaked in oil for the required period. The difference in weight gives the weight of oil absorbed after soaking. The results, expressed as a percentage of their dry weights, are presented in table (4.23) and related to other HCP and Mortar properties as shown in the following section.

4.4.11.2 Absorption Characteristics of HCPs and Mortar.

In Figs. (4.40, 4.41, 4.42, 4.43), the oil and water absorbed during a soaking period of 300 days and water loss at

Mix Designation	Curing condition	$a \times 10^3$	b	r
A	600-days SW 600-days SO 600-days SE	5.067 8.590 6.192	-0.253 -0.293 -0.270	99.99 99.89 99.19
B	600-days SW 600-days SO 600-days SE	1.938 5.299 4.103	-0.217 -0.296 -0.277	99.77 99.94 99.69

Table (4.22) Constants of regression-equation (4.14) Compressive strength Vs T-porosity.

Mix Designation	w/c Ratio	% of water lost during drying at 105°C	Oil Absorbed % by Wt.			Degree of Saturation*		
			Soaking Period (days)					
			1	140	300	1	140	300
HCPs	0.30 0.35 0.40	16.70 20.20 23.20	0.40 0.86 1.37	8.60 11.40 14.84	10.70 13.90 16.90	2.69 4.78 6.64	57.90 63.40 71.90	72.10 77.30 81.90
1:2 Mortar	0.40 0.50 0.60	8.50 10.70 12.40	0.37 0.75 0.96	5.40 7.50 9.20	6.20 8.30 9.90	4.88 7.75 8.78	70.20 77.00 84.30	80.60 85.20 90.70
1:3 Mortar	0.50 0.60 0.70	7.40 9.10 10.60	0.76 1.14 1.50	4.78 6.55 7.89	6.10 7.50 8.85	11.20 14.04 16.10	70.22 80.34 84.22	89.60 92.00 93.90

$$* \text{ Degree of saturation} = \frac{\text{Vol. of oil absorbed}}{\text{Vol. of water lost during drying}} \times 100$$

Table (4.23) Oil Absorbed by HCP and Mortar Cubes after Different Soaking Periods.

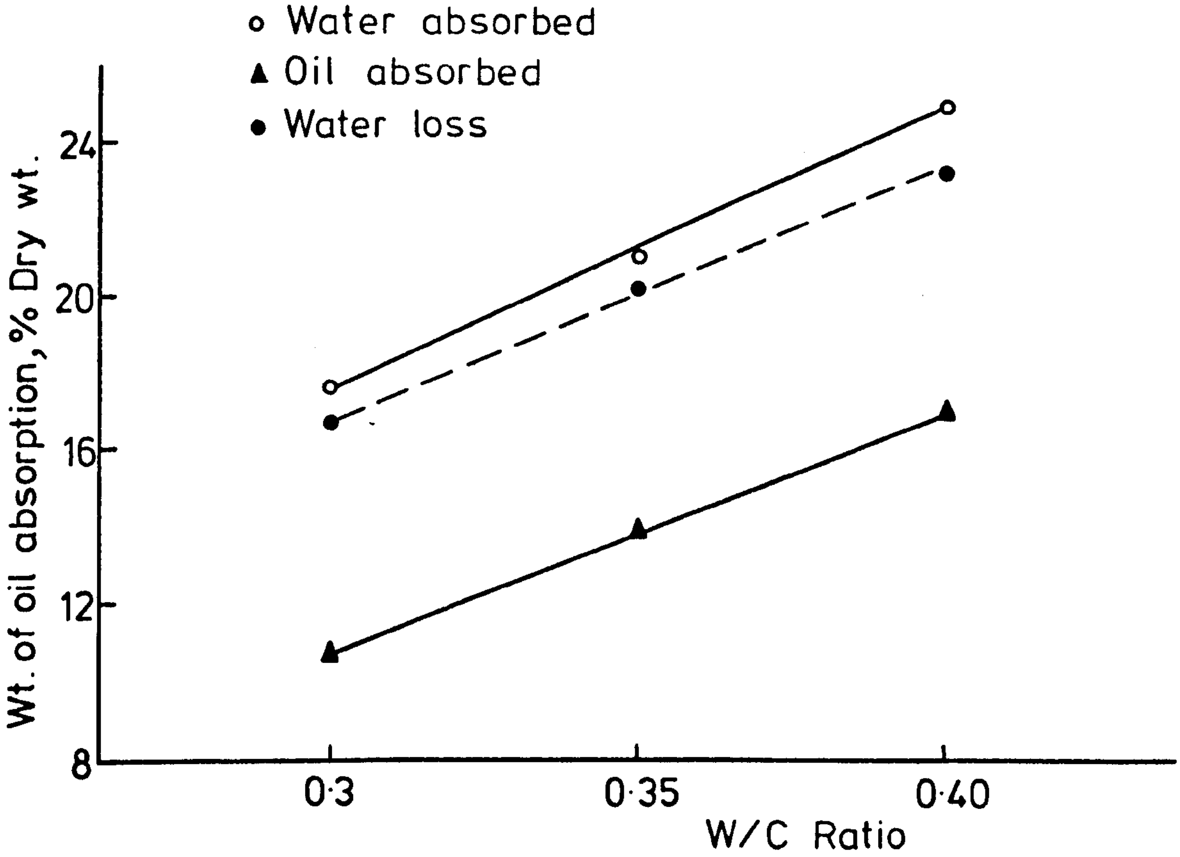


FIG. 4-40. WEIGHT OF ABSORPTION AND WATER LOST ON DRYING TO 105°C OF H.C.P's

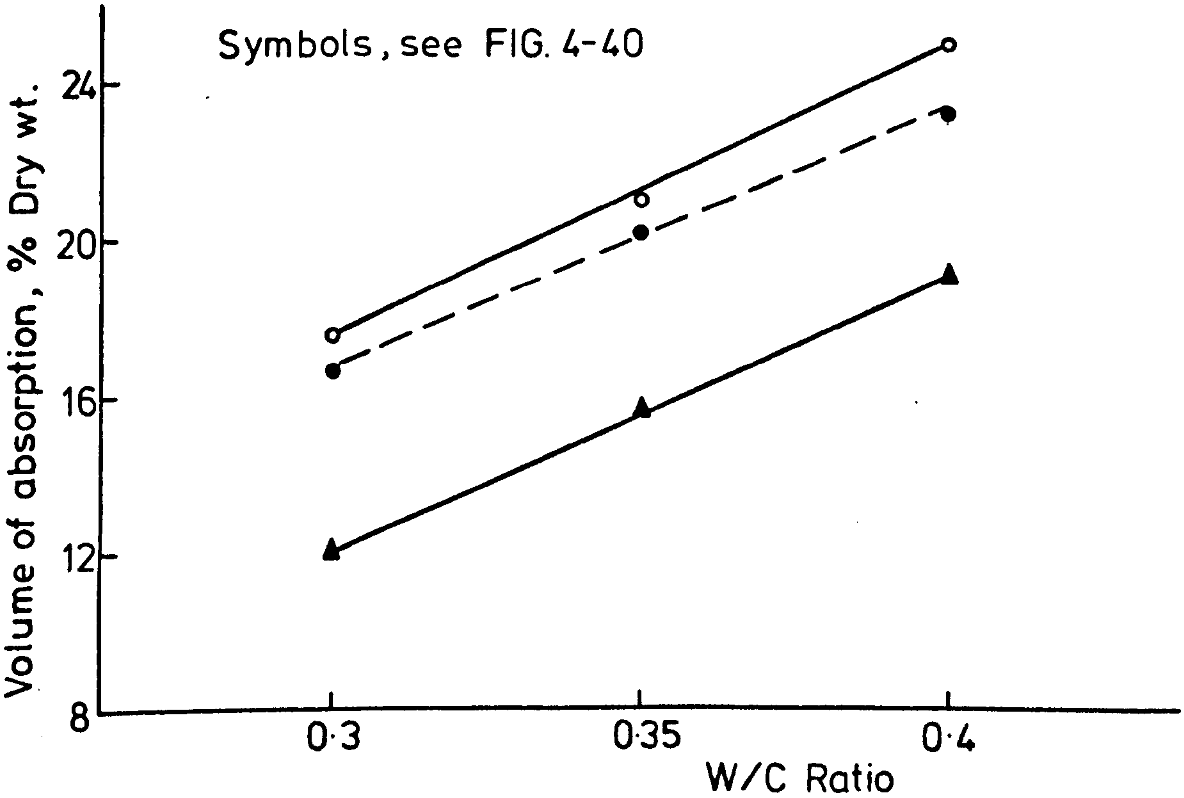


FIG. 4-41. VOLUME OF ABSORPTION AND WATER LOST ON DRYING TO 105°C OF H.C.P.'s

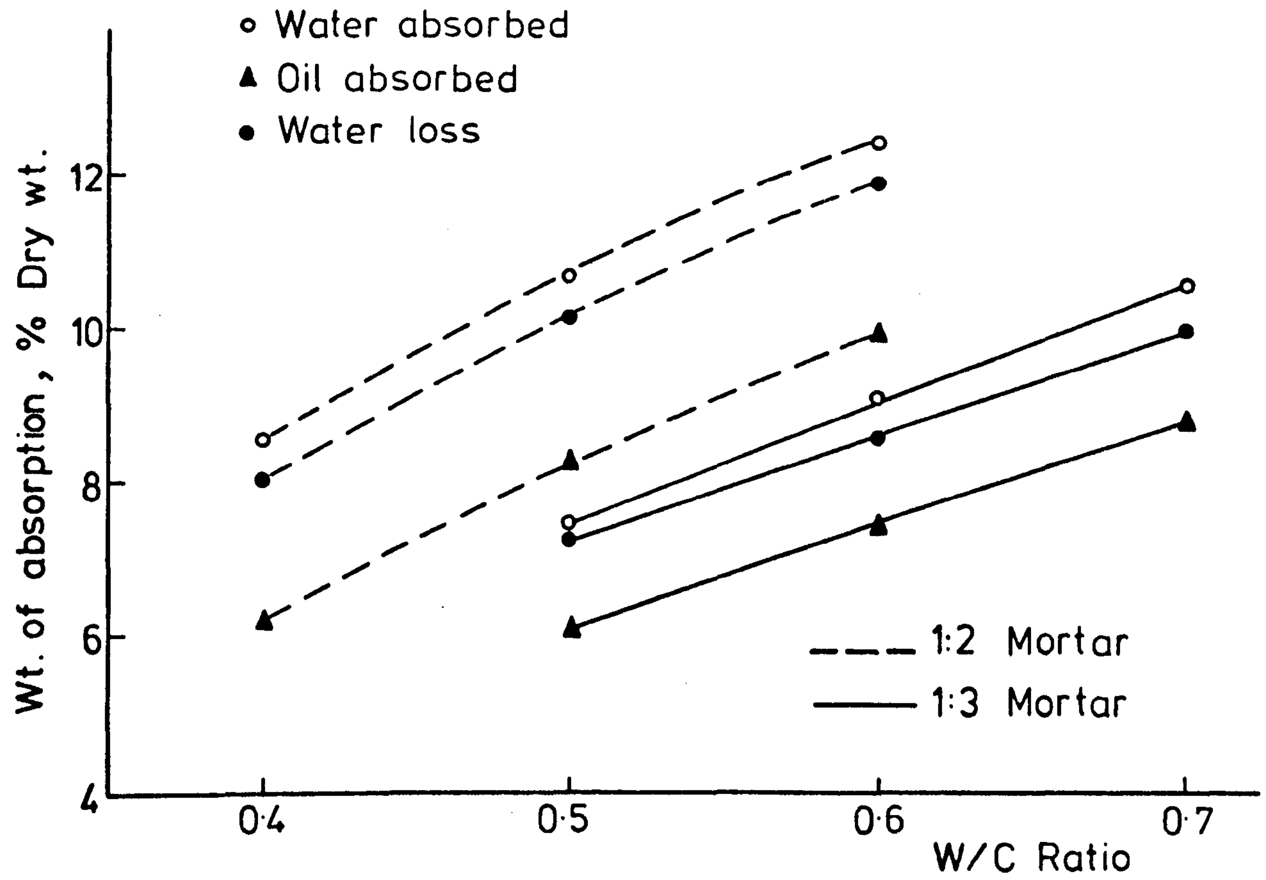


FIG. 4-42. VARIATION OF ABSORPTION AND WATER LOSS DURING DRYING FOR MORTAR SPECIMENS

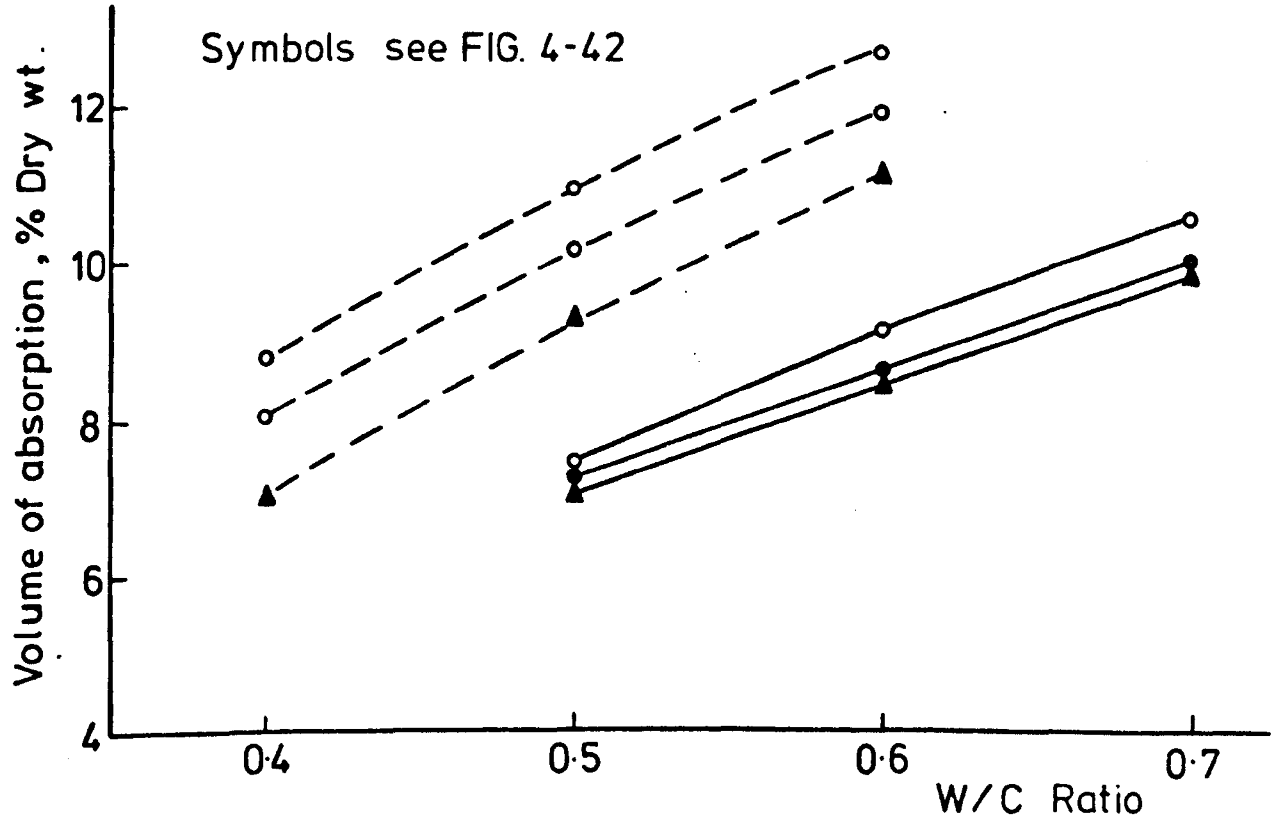


FIG. 4-43. VOLUME OF ABSORPTION AND WATER LOSS DURING DRYING FOR MORTAR SPECIMENS

105° C for the various w/c ratios are illustrated for HCPs, 1:2 Mortars, and 1:3 Mortars. It is observed that:-

1. The test results illustrated in figs. (4.40) to (4.43) show that the amount of oil absorption is linearly related to the w/c ratio and can be expressed by the following formula:-

$$A = a\,(w/c) + b \qquad \ldots\ldots \qquad (4.15)$$

where A is the absorption capacity (gm/gm) x100 (% dry weight) and the constants a and b were determined and given in table (4.24) for HCPs and Mortars.

2. In figs. (4.41, 4.43), both the water loss on drying and oil absorption are replotted after dividing by the appropriate specific gravity. It is clear that the volume of oil absorbed was less than the volume of water lost during drying by 18-28% for HCPs, 10-18% for 1:2 Mortars, and 6-9% for 1:3 Mortars. This means that full saturation was never achieved and some voids left by water loss were unfilled by the oil. This might be due to the presence of large molecules (up to 15 Å) of crude oil which would clog up the narrow-neck entry pores and thus prevent full saturation.

3. For HCPs, the amount of absorbed water was higher than the amount of water lost during oven drying. This confirms the fact(63) that drying causes some structural damage in the hardened cement paste specimens and this creates new cracks which increases the absorption capacity. For mortar specimens, the volume of absorbed water was between 3-6% less than the volume of water lost during drying. This means that the drying process creates more structural damage in HCPs than in Mortar specimens.

4. Figs. (4.44, 4.45) show the relationship between absorption and total porosity which can be expressed by the following linear equation:-

$$A = a P - b \qquad \ldots\ldots \qquad (4.16)$$

The constants a and b are given in table (4.25) for both HCP and Mortar.

4.4.12 Absorption Characteristics of Plain Concrete.

4.4.12.1 Variation of Oil Absorption with Time.

The amount of absorbed oil was measured using the gravimetric method as described in section (4.5.11.1). The results, expressed as a percentage of their dry weight are presented in table (4.26) and plotted against soaking period as shown in figs. (4.46, 4.47). The following general observations are made:

1. When the dried speciems were soaked in crude oil, the absorption started initially at a high rate which slowed down with time. Most of the absorption occurred within the first six months of storage in oil. The relationship between the amount of absorbed oil and the soaking period for the test results presented in figs. (4.46, 4.47) can be expressed by the following formula:

$$A = a \ln t + b \qquad \ldots\ldots \qquad (4.17)$$

The constants a and b are given in table (4.27) for all concrete mixes.

2. After a 24 hours soaking period, the weight of absorbed oil, varied between 5-10% as a percentage of the weight of water lost during drying. The weight of water absorbed by the dry

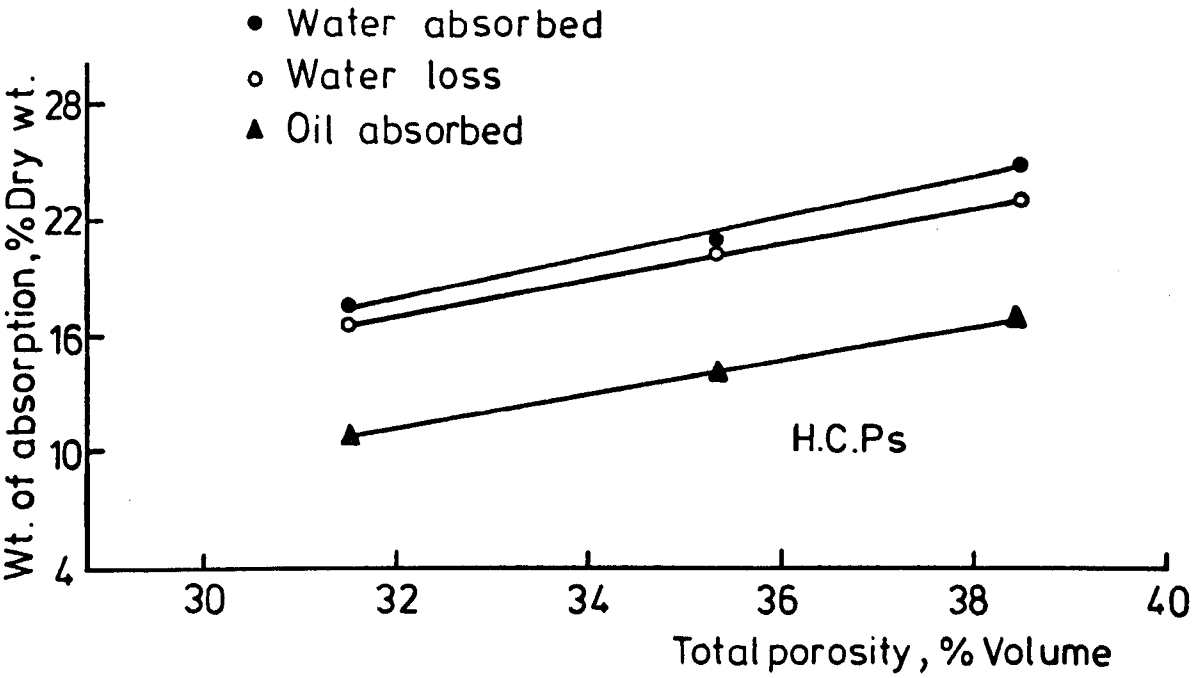


FIG. 4-44 VARIATION OF ABSORPTION WITH TOTAL POROSITY OF H.C.Ps

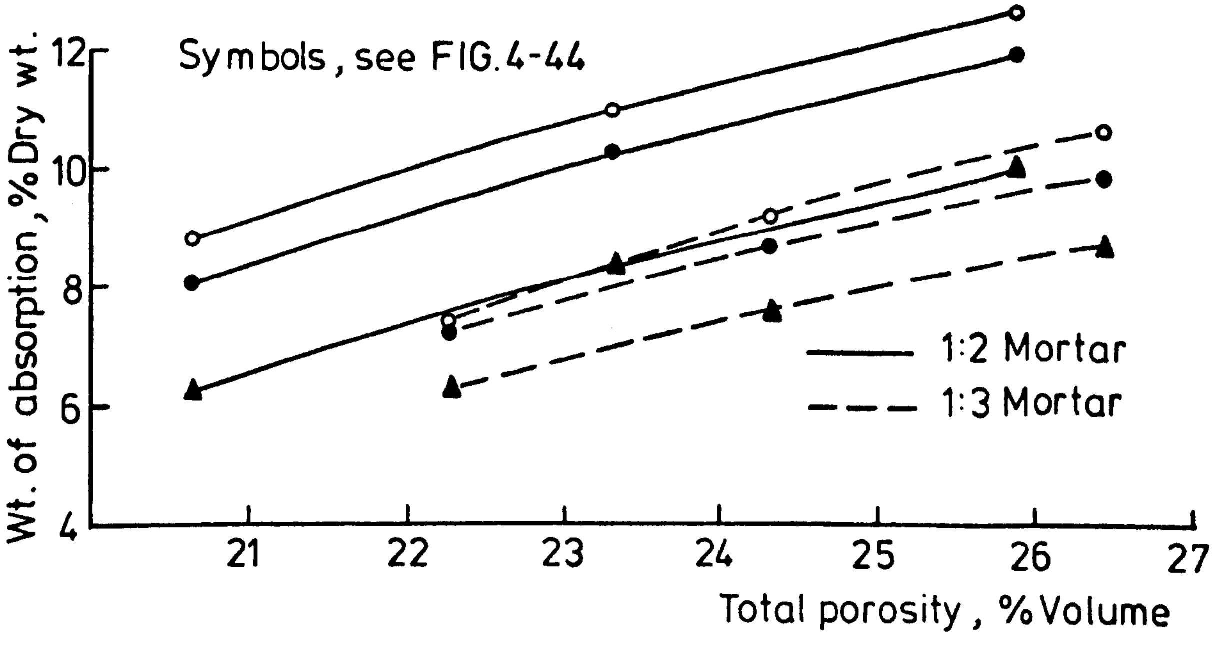


FIG. 4-45 VARIATION OF ABSORPTION WITH TOTAL POROSITY OF MORTAR

Materials	Type of absorption	a	b	r
HCP	Oil Water	63 66	8 3	99.9 99.88
1:2 Mortar	Oil Water	18.5 19.3	- 1.10 + 0.41	99.82 99.80
1:3 Mortar	Oil Water	11.8 12.5	+ 0.45 + 1.00	99.87 99.91

Table (4.24) Constants of regression (4.15)- Absorption Vs. w/c Ratio

Materials	Type of absorption	a	b	r
HCP	Oil Water	0.95 1.00	19.5 14.5	99.58 99.93
1:2 Mortar	Oil Water	0.70 0.73	8.0 7.0	99.93 99.92
1:3 Mortar	Oil Water	0.56 0.60	6.2 6.0	99.84 99.89

Table (4.25) Constants of regression (4.16)- Absorption Vs.Total Porosity.

Mix Designation	w/c Ratio	% of water lost during drying	14 - days			35 - days			150 - days			280 - days			600 - days		
			Oil Content % Dry Weight	Average depth of penetration (mm)	Area reached by oil (cm^2)	Oil Content % Dry Weight	Average depth of penetration (mm)	Area reached by oil (cm^2)	Oil Content % Dry Weight	Average depth of penetration (mm)	Area reached by oil (cm^2)	Oil Content % Dry Weight	Average depth of penetration (mm)	Area reached by oil (cm^2)	Oil Content % Dry Weight	Average depth of penetration (mm)	Area reached by oil (cm^2)
A1	0.4	6.0	0.53	9	34	2.65	17	57	4.15	31	80	4.65	38	94	5.00	41	97
A2	0.5	6.20	0.55	12	40	3.03	19	65	4.70	32	84	5.04	39	95	5.49	43	97
A3	0.6	6.45	0.59	14	46	3.90	21	70	5.15	33	85	5.40	40	96	5.65	45	98
B1	0.4	5.37	0.39	8	33	2.39	15	61	4.05	32	81	4.38	37	95	4.68	42	97
B2	0.5	5.62	0.45	10	39	2.80	17	68	4.51	33	84	4.84	39	95	4.98	44	98
B3	0.6	6.00	0.49	12	45	3.25	19	72	4.85	35	85	5.28	40	96	5.41	46	98

Table (4.26) Oil absorption (% by weight), Variation of area saturated by oil, and the average depth of penetration with time and water/cement ratio (using prism of 100x100x500 mm).

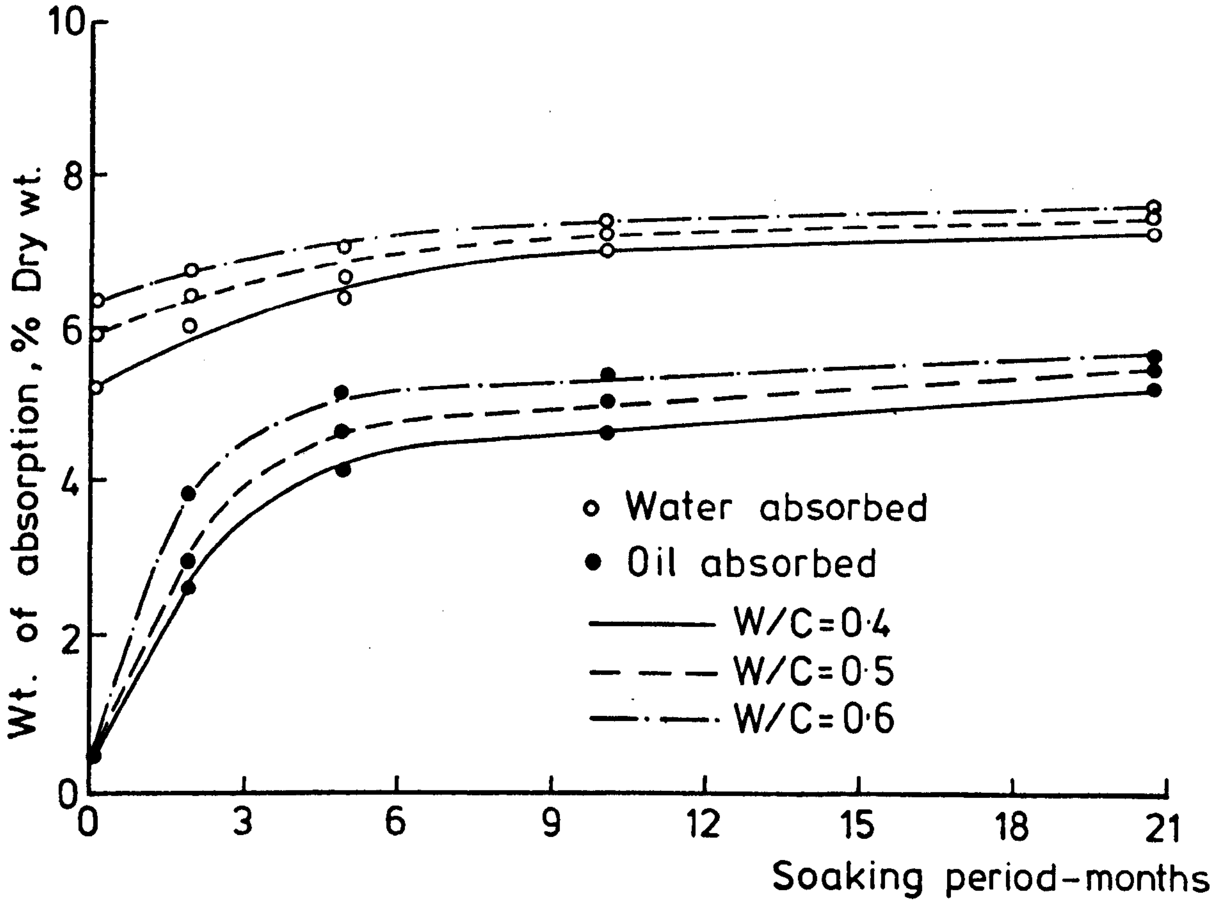


FIG. 4-46. VARIATION OF ABSORPTION WITH SOAKING PERIOD FOR CONCRETE OF 10mm MAX. SIZE OF AGGREGATE

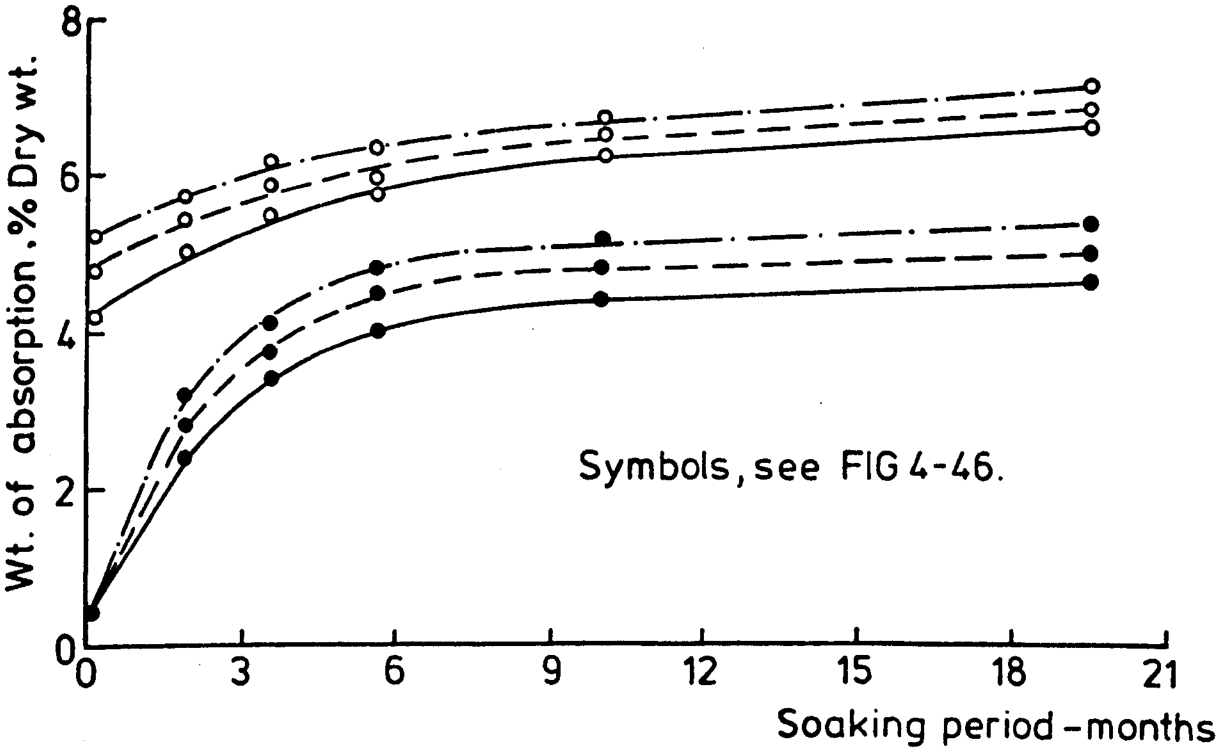


FIG. 4-47. VARIATION OF ABSORPTION WITH SOAKING PERIOD FOR CONCRETE OF 20mm MAX. SIZE OF AGGREGATE

specimens for the same soaking period varied between 90-95% as a percentage of the water lost during drying. But after a soaking period of 600 days, the weight of absorbed oil as a percentage of the water lost during drying increased to between 83-88% for Mix A and 87-90% for Mix B. The concrete of higher w/c ratio had a higher porosity and therefore had the highest amount of absorption.

4.4.12.2 The Variation of Depth of Oil Penetration with Time.

The Ultra-Violet lamp technique was used to measure the depth of penetration of absorbed oil. This technique depends on the fact that hydrocarbons change in colour as they absorb ultra-violet light. In this test, portions of oil soaked concrete, taken from the specimens which had been tested for strength at different soaking periods, were placed under the ultra-violet lamp. A change in colour of the concrete areas containing oils was observed and approximately defined and marked as shown in plates (4.1, 4.2, 4.3), and then measured to $\pm 0.1\ \text{cm}^2$ using a planimeter after plotting the boundary on drawing paper. The average depth of oil penetration was also measured and these results are shown in table (4.26) and fig. (4.48). The following is observed:-

1. The specimens removed from the oil after different soaking periods showed that the depth of penetration increased rapidly at an initially high rate and then continued at a slower rate. The relationship between the depth of penetration (D) and soaking period (t) can be expressed by the following formula:

$$D = a\,t^{b} \qquad \ldots\ldots \qquad (4.18)$$

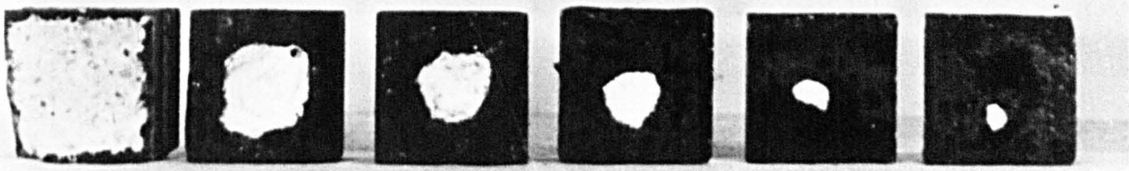


PLATE. 4-1. PENETRATION OF OIL THROUGH THE CONCRETE CUBES AS SHOWN UNDER U.V. LAMP

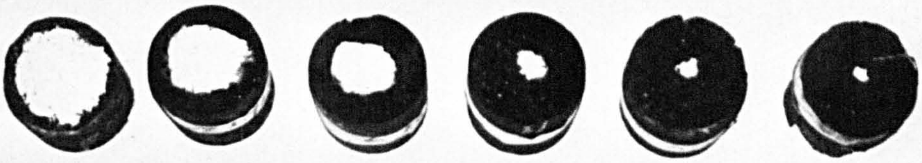


PLATE. 4-2. PENETRATION OF OIL THROUGH THE CONCRETE CYLINDER AS SHOWN UNDER U.V. LAMP

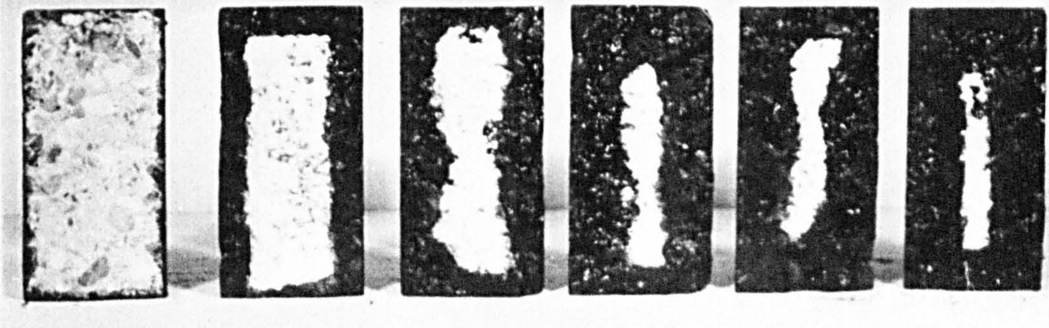


PLATE. 4-3. PENETRATION OF OIL THROUGH THE CONCRETE CYLINDER (SPLITTING TEST) AS SHOWN UNDER U.V. LAMP

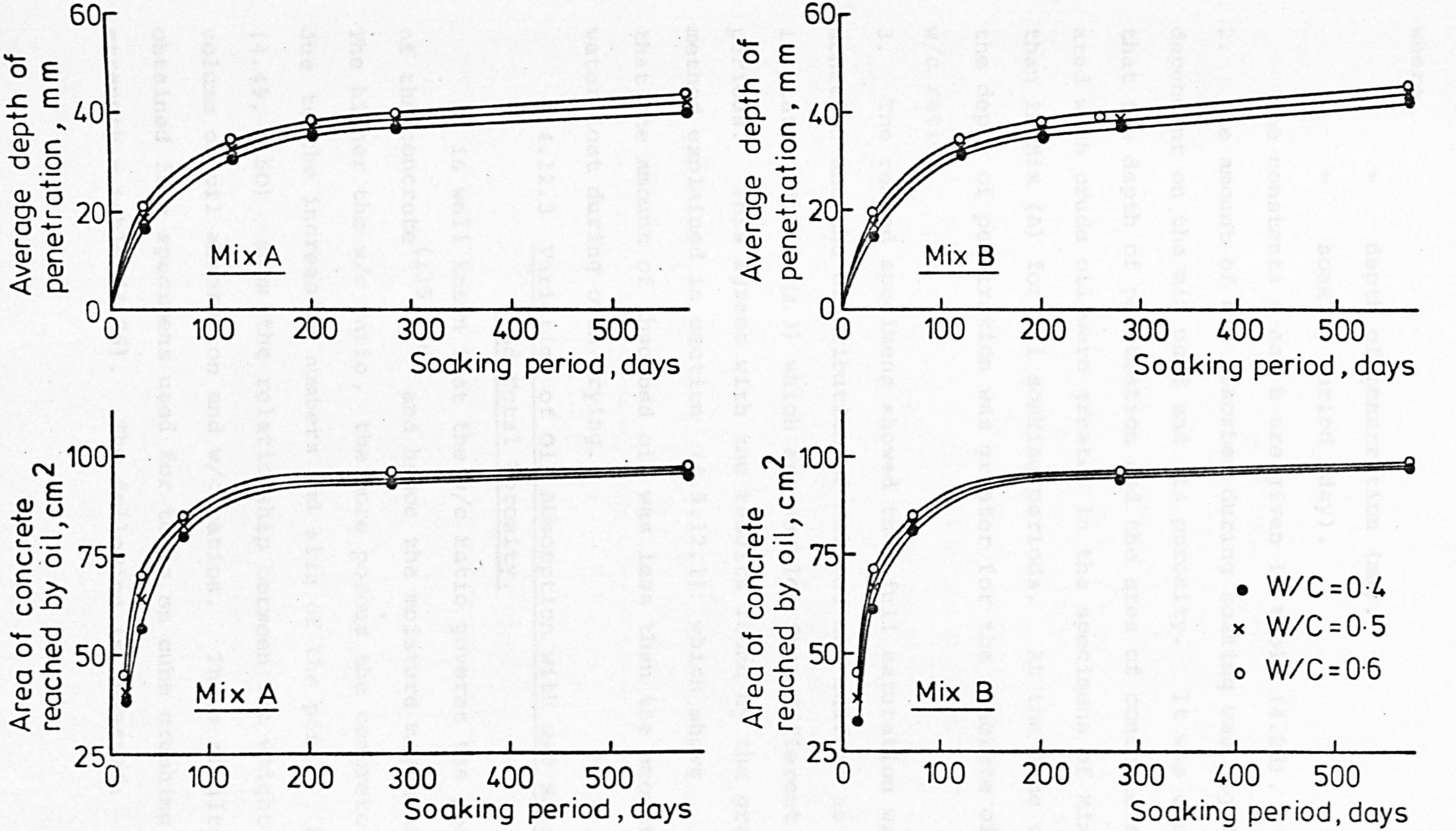


FIG. 4-48 AVERAGE DEPTH OF PENETRATION AND AREA REACHED BY OIL FOR CONCRETE CUBES OR PRISMS, 100 × 100 mm CROSS-SECTION, Vs. SOAKING PERIOD

where

D = depth of penetration (mm).

t = soaking period (day).

The constants a and b are given in table (4.28).

2. The amount of oil absorbed during soaking was highly dependent on the mix used and its porosity. It was clear that the depth of penetration and the area of concrete saturated with crude oil were greater in the specimens of Mix (B) than in Mix (A) for all soaking periods. At the same time, the depth of penetration was greater for the concrete of higher w/c ratio.

3. The tested specimens showed that full saturation was never achieved and the distribution of oil was not uniform as shown in plates (4.1) to (4.3) which are samples from different soaking periods. This agrees with the results found by the gravimetric method explained in section (4.5.12.1) which shows that the amount of absorbed oil was less than the amount of water lost during oven drying.

4.4.12.3 Variation of Oil Absorption with w/c Ratio and Total Porosity.

It is well known that the w/c ratio governs the porosity of the concrete[135,136] and hence the moisture movement. The higher the w/c ratio, the more porous the concrete will be due to the increasing numbers and size of the pores. Figs. (4.49, 4.50) show the relationship between the weight and volume of oil absorption and w/c ratios. These results were obtained from specimens used for tests on cube crushing strength - table (4.26). The following is observed:

Mix Designation	w/c Ratio	Soaked in oil (SO)			Soaked in water (SW)		
		a	b	r	a	b	r
A	0.4	0.73	0.40	99.40	0.30	5.2	98.2
	0.5	0.79	0.50	99.50	0.25	5.80	97.8
	0.6	0.85	0.60	98.90	0.20	6.30	98.1
B	0.4	0.70	0.30	98.90	0.36	4.17	97.1
	0.5	0.76	0.40	99.10	0.32	4.71	96.5
	0.6	0.82	0.50	99.50	0.28	5.21	96.9

Table (4.27) Constants of regression - eqn. (4.17)
Absorption Vs. time.

Mix Designation	w/c Ratio	a	b	r
A	0.4	6.0	0.30	95.72
	0.5	7.5	0.28	97.63
	0.6	9.0	0.26	98.15
B	0.4	5.0	0.34	96.20
	0.5	6.0	0.32	96.25
	0.6	7.0	0.30	97.38

Table (4.28) Constants of Regression - Eqn. (4.18)
Depth of Penetration Vs. Time.

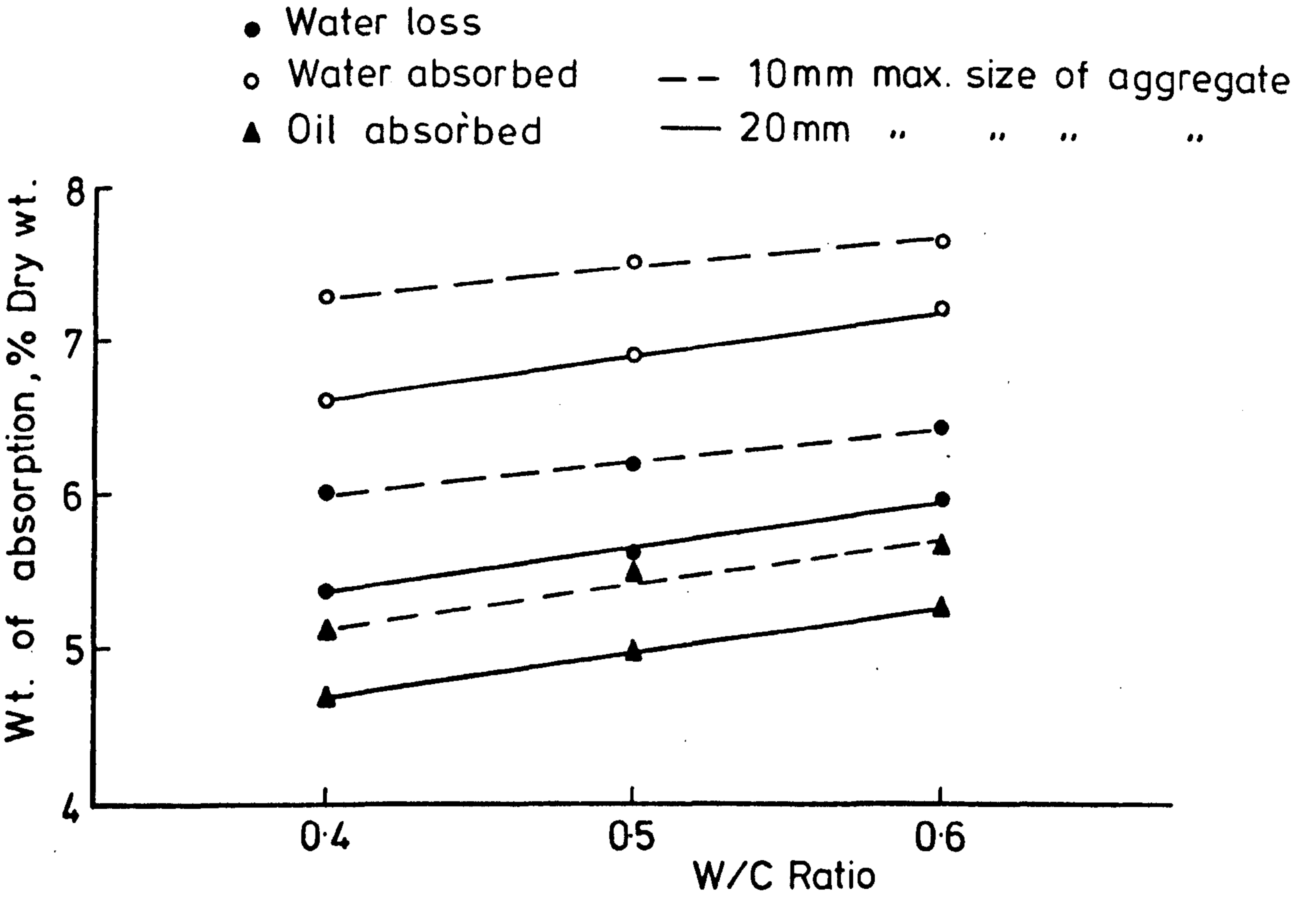


FIG. 4-49. ABSORPTION AFTER 600 DAY RESOAKING AND WATER LOST BY DRYING AT 105°C FOR CONCRETE SPECIMENS

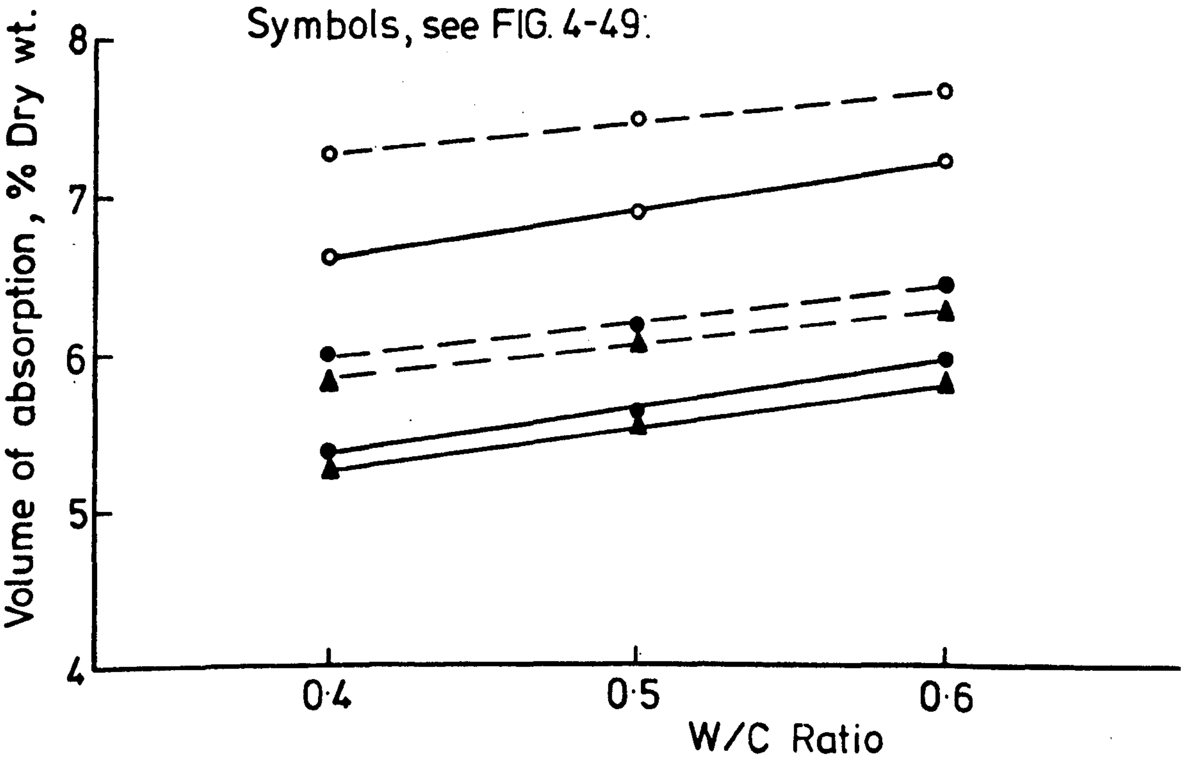


FIG. 4-50. VOLUME OF ABSORPTION AFTER 600 DAY RESOAKING AND WATER LOST BY DRYING AT 105°C FOR CONCRETE SPECIMENS

1. The amount of absorption increased with the increase of w/c ratio. The concrete of Mix A absorbed more than the concrete of Mix B because of its higher porosity.

2. The amount of water absorbed was higher than the amount of water lost on drying to 105^{o} C partly because extra cracks may have been created during the drying and cooling processes.

3. The volume of crude oil absorbed per gram of dry concrete is plotted in fig. (4.50) and compared with the amount of water lost on drying to 105^{o} C and on water absorbed for the same soaking period. It is clear, as for hardened cement pastes and Mortars, that less than 100% of voids were filled by oil, possibly for reasons explained in section (4.5.11.2).

4. The total porosity and the amount of oil absorption are given in tables (4.21) and (4.26) for all the concrete mixes. The relationship between them is illustrated in fig. (4.51) which shows that the higher the total porosity the higher the absorption. Their relationship can be expressed by the following formula:

$$A = A_o P_c^{a} \qquad ..... \qquad (4.19)$$

where A_o is the absorption of concrete of zero porosity and (a) is a constant. A_o and a were determined by a regression analysis and are given in table(4.29).

Type of absorption	A_o	a	r
water absorption	1.04	0.69	99.40
oil absorption	0.42	0.90	96.80

Table (4.29) Constants of Regression - equation (4.19) Amount of absorption Vs. porosity.

4.5 DISCUSSION OF THE TEST RESULTS.

Test results presented in the previous sections have shown that the strength and elastic modulus of HCP, Mortar and Concrete saturated with crude oil were below those tested in a water saturated condition and were considerably below those dried and sealed as a reference in the laboratory. The effect of saturation with oil on the studied properties of cement paste, mortar, and concrete and its relation with the effect of other factors such as drying in the oven at 105° C, saturation of the dry specimens with water, effect of porosity and size of aggregate are discussed in this section.

4.5.1 Effect of Drying on the strength of Concrete.

It has been stated(65) that the moisture content of concrete test specimens has an important influence on its strength. The drying process influences the physical and chemical bonds of the concrete. However, the change in strength also depends on the drying condition and the type of the test.

As mentioned earlier - sections (4.3.2, 4.4.1), there was an increase in the compressive strength of the oven-dried cement paste, mortar, and concrete. The specimens were moist cured for 7-days to allow significant strength gain, and they were then dried at 105° C for about 14 days until weight equilibrium was reached. The oven temperatures were raised or lowered at a rate not greater than 8° C/hour to try and avoid thermal cracking of the specimens. In this case, the slow drying process at a higher temperature causes an increase in the rate of movement of capillary water by raising the internal pressure and also causes an increase in the rate of hydration.(137)

The speed up in the chemical reactions of hydration will increase the early strength of concrete. The other reason might be due to the increase in van der Waals force of attraction as the water evaporates, thus drawing the particles together thereby causing an increase in the strength and rigidity of concrete(31,138).

The effect of drying on the tensile splitting strength was less than the effect on the compressive strength - section 4.4.2 (table 4.10). There was an insignificant ($P \leqslant 0.05$) increase which varied between 2 - 2.5% in the tensile splitting of the concrete specimens after being oven-dried. On the other hand the modulus of rupture of the oven-dried specimens showed a significant increase as described in section (4.4.4).

Results of tests carried out by Pihlajavaara(64), Hansen(139). and Johnston and Sidwell(65) are in agreement to a large extent with the results achieved by the present investigation, despite some variation in the drying process. Johnston and Sidwell's work states that prolonged drying of concrete results in slightly increases tensile and compressive strength, and gives the main reason as the avoidance of wedge action by adsorbed water in the cement gel. Rapid drying of plain concrete is accompanied by shrinkage of the drying surface against the stable saturated interior and cracks will form from the surface causing a significant decrease in tensile strength. Compressive strength is relatively insensitive to rate of drying.

Pihlajavaara(64) concluded that drying causes strengthening and wetting creates a weakening of porous media. He showed that the ratio of wet strength to dry strength of various concretes seems to vary 1:1.2 to 1:2. Hansen's specimens(139) exhibited slightly increased compressive strength after 21 days drying at different levels of relative humidity, this increase was probably related to the wedge action of absorbed water. Walker and Bloem(68) found that the flexural strength test is sensitive to the variation in curing, and to the moisture condition of the specimens. It was observed that in the case of the test specimen which had been allowed to dry very slowly, so that internal stresses were redistributed and alleviated by creep, an increase in the measured flexural strength was obtained.

The sealed reference concrete specimens showed a continuous reduction in the compressive strength with time, as shown in section (4.4.1). This reduction might be caused by internal cracks of solid structure during drying and these cracks will propagate slowly with an unstable length during cooling. The formation and propagation of such cracks have long been recognized as the cause of the fracture and failure of concrete(163).

With regard to the sealed specimens, the tensile tests showed much the same results as the compressive tests, as shown in figs. (4.20 to 4.23). One can presume that these results occur for the same reasons, such as crack development in the cement matrix and at the cement-aggregate interface during cooling.

4.5.2 The Effect of Oil Saturation on the Cube Crushing Strength of HCP, Mortar and Concrete.

The effect of crude oil saturation on the mechanical properties of HCP, mortar, and concrete can be observed by comparing the same properties with water saturated and sealed specimens. There was a sharp reduction in the compressive strength of dry specimens after 24 hours of soaking in water. The immersion of a companion specimen in crude oil, on the other hand, produces no immediate effect on the compressive strength. This might be explained by the very slow penetration of oil in concrete compared to the fast penetration of water. The absorption of water into the hardened cement paste and concrete pores is known to result in the dilation of the cement gel thereby decreasing the cohesive forces between the cement particles and reducing the strength(140). In the present investigation, the dry concrete specimens absorbed crude oil up to 0.5% of their dry weight (degree of saturation of 8-10%) compared to about 6% of water (degree of saturation 85-95%) within a soaking period of 24 hours. This small amount of absorbed oil is not expected to cause any weakening effect.

From a microstructural view point, it has been suggested(141,142,143) that in brittle materials such as hardened cement paste, mortar, and concrete, it is possible to relate the strength reduction to the surface energy. On saturation with fluids, the surface energy decreases and hence reduces the strength, and the amount of reduction can be related to the molecular size of the fluid adsorbed. Crude oil content of 0.5% of dry weight and an average depth of penetration between 3-5 mm as shown in figs. (4.46) to (4.48), would not cause a considerable reduction in the surface energy.

The compressive strength of dry specimens kept continuously soaked in oil was reduced proportionately to the soaking period due to the increasing amount of absorbed oil, as expressed in the test results presented in fig. (4.52). This reduction is also expressed in terms of the percentage of the 1-day sealed strength, as shown in table (4.30). The reduction in the compressive strength was clear for all specimens soaked in oil and it increases with increasing w/c ratio and soaking period.

The reduction in strength of concrete due to crude oil saturation is illustrated in fig. (4.53) which is a plot of the percentage decrease in strength for samples stored in crude oil over the oven dry strength against the weight of oil absorbed. Though a considerable scatter of results is obtained, it is evident that the strength reduction was dependent upon the amount of oil absorbed. The continued reduction of strength with time is due to the continued absorption of oil into the pores and microcracks of the concrete resulting in the continued reduction of the cement paste-aggregate bond and hence the compressive strength. Table (4.31) shows the numerical and percentage strength reduction of concrete specimens averaged over a monthly period. It can be seen that the rate of strength loss decreases with time. Absorption of oil reached an almost constant rate after the 10th month of oil soaking - figs. (4.46, 4.47) and consequently the reduction of strength of concrete was found to be very small after that time. Chemical, microstructural and physical phenomena might explain this strength reduction.

From a chemical viewpoint, mineral oils(9) do not attack matured concrete. These oils contain no constituents which react chemically with set concrete, provided they are free

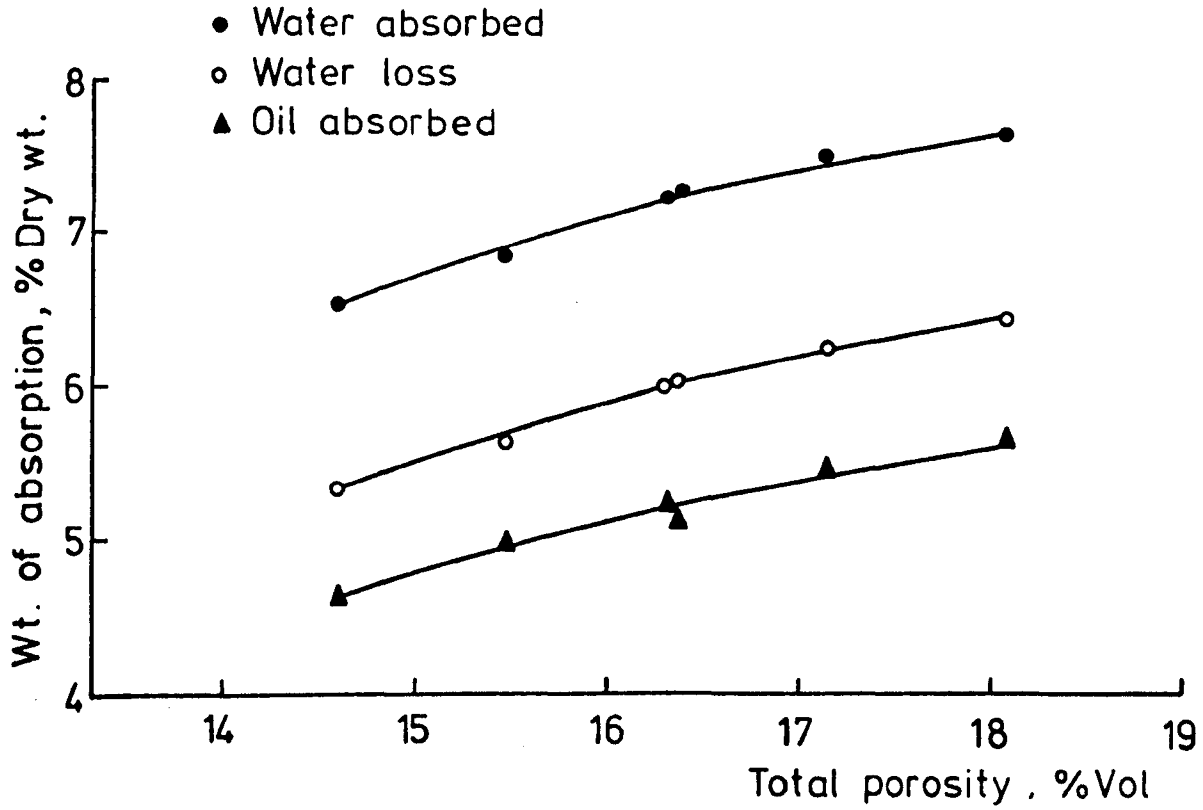


FIG. 4-51. VARIATION OF ABSORPTION WITH TOTAL POROSITY FOR CONCRETE SPECIMENS

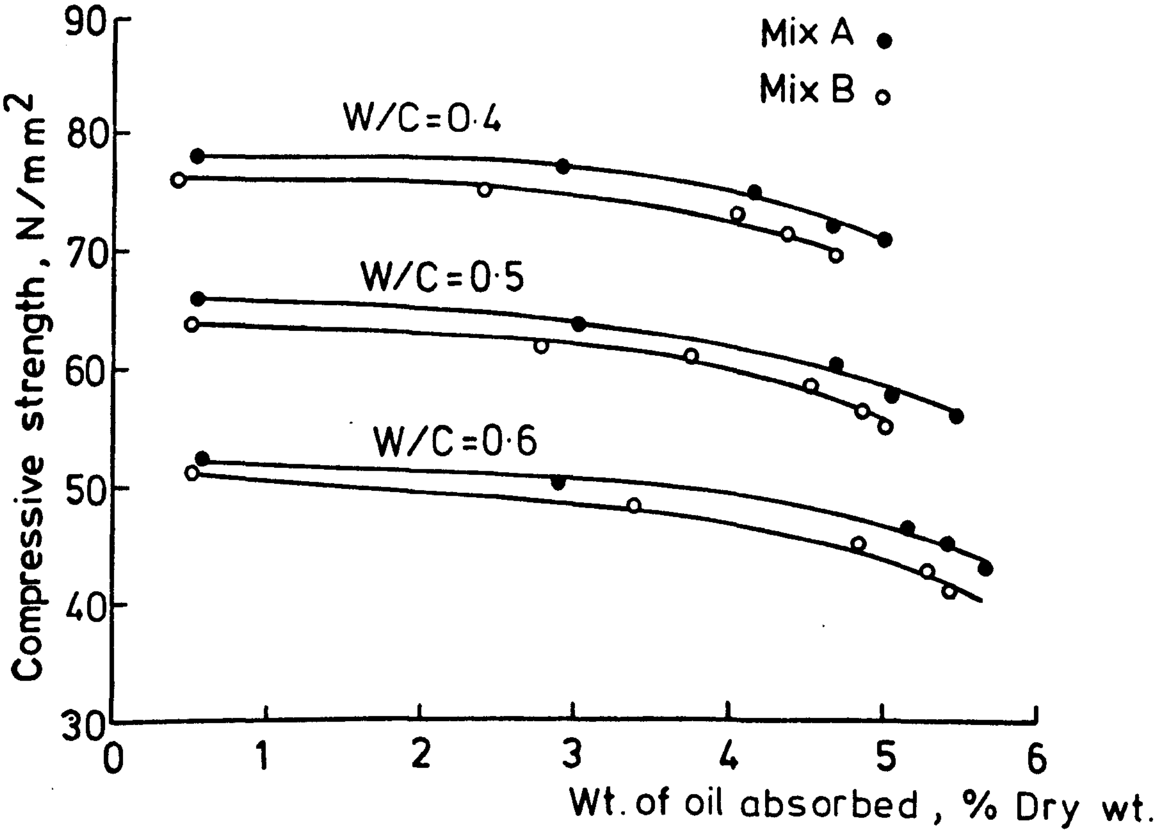


FIG. 4-52. THE RELATIONSHIP BETWEEN THE COMPRESSIVE STRENGTH OF OIL SOAKED CONCRETE AND OIL ABSORPTION

Mix Designation	w/c Ratio	1 - day			70 - day			160 - day			300 - day			600 - day		
		Sealed (SE)	Soaked in oil (SO)	Soaked in water (SW)	Sealed (SE)	Soaked in oil (SO)	Soaked in water (SW)	Sealed (SE)	Soaked in oil (SO)	Soaked in water (SW)	Sealed (SE)	Soaked in oil (SO)	Soaked in water (SW)	Sealed (SE)	Soaked in oil (SO)	Soaked in water (SW)
A	0.4	100	98.7	74.7	98.2	97.5	86.7	96.2	94.7	93.7	93.6	91.1	99.4	91.8	90.5	104.4
	0.5	100	98.5	65.7	96.3	94.8	76.9	90.7	88.8	85.8	87.3	85.8	91.8	86.3	84.3	100.3
	0.6	100	97.2	63.9	94.4	92.6	78.9	89.8	86.1	85.2	86.1	82.4	90.7	83.3	79.6	103.3
B	0.4	100	98.4	73.5	98.1	96.8	87.1	96.8	94.2	95.3	94.2	92.3	98.7	92.3	89.7	104.3
	0.5	100	97.7	66.2	95.4	93.8	86.9	90.8	90.0	94.6	89.2	86.9	97.7	87.7	83.8	104.1
	0.6	100	97.2	63.2	95.3	91.5	85.8	90.2	86.4	91.5	84.0	80.2	97.2	83.0	77.4	103.0

Table (4.30) Cube crushing strength as a % of the 1-day Sealed Strength.

Mix Designation	w/c Ratio	10th Week		23rd Week		43rd Week		36th Week	
		Reduction[+] N/mm^2	% of total Reduction	Reduction[++] N/mm^2	% of total Reduction	Reduction[*] N/mm^2	% of total Reduction	Reduction[**] N/mm^2	% of total Reduction
A	0.4	0.86	11.5	0.83	11.1	0.43	5.7	0.10	1.3
	0.5	1.50	14.3	1.33	12.7	0.43	4.1	0.10	1.0
	0.6	1.71	15.6	1.33	12.1	0.43	3.9	0.10	0.9
B	0.4	0.94	11.8	0.80	10.0	0.43	5.4	0.15	1.9
	0.5	1.50	14.3	1.00	9.5	0.43	4.1	0.20	1.9
	0.6	1.93	16.1	1.00	8.3	0.54	4.5	0.20	1.7

[+] Reduction and % Reduction over 1-day oven dry strength.

[++] Reduction and % Reduction over 10th week oil sat.

[*] Reduction and % Reduction over 23rd week oil sat.

[**] Reduction and % Reduction over 43rd week oil sat.

Table (4.31) Reduction in Compressive Strength (per month) due to Crude Oil Saturation.

from any impurities and are of mineral origin, containing no fatty acids or greasy materials. Crude oil is a mixture of such substances. The sulphate, for example, may effect the strength due to chemical reaction, but from the crude oil analysis (Chapter 3), the sulphate content was about 2% which is too small to have a serious effect in such a short time.

From the microstructural viewpoint, the solid surface in cement gel is subjected to forces of attraction known as van der Waals. When the paste is thoroughly dried out, these forces and, to some extent, chemical bonds are the only bonds present between the gel particles. With the space filled with water, the water molecules act against the above mentioned bonds between adjacent surfaces, thus forcing them apart and weakening the structure. Cooke and Hague(144) have shown that benzene and parafin with molecular sizes of about 2 and 3 times greater than water, do not reduce the strength of concrete, but ethyl alcohol with a molecular size only 50% greater than water, reduces the strength to the same degree as water molecules. Hannant(145) found that liquid with large molecular sizes has no effect on creep but methyl alcohol has some effect. Since a molecule of methyl alcohol is about 3.5 Å in diameter, i.e. approximately the same as water, it would suggest that molecular size is a significant factor. Crude oil has molecular sizes 3 times lower and up to 6 times higher than water and the absorption of oil might have the same effect as water molecules, so that the presence of oil would alter the force-field between the solid particles, and it could exert forces on the confining surfaces even if the oil did not entirely fill the space.

Physically,(38,43,45,46,47) the effect of the testing machine and the type of failure occurring in the tested specimens could contribute to the strength reduction. Examination of the oil soaked and sealed cube specimens which were tested under normal conditions of end restraint showed that after failure, there were zones of sound concrete in the shape of pyramids at each end of the specimens, as shown in plate (4.4). This type of failure is called pseudo-shear failure because it involves simultaneous splitting and shearing. The shearing resistance of concrete is usually considered(34,35) as the sum of cohesive strength plus the internal frictional resistance of concrete along the plane of sliding. The cohesive part of the compressive shear bond is lost when the strength of interface is exceeded, and the sliding is resisted thereafter only by the friction. The presence of oil in the cracks between aggregate and mortar, and along the sliding planes would reduce the coefficient of friction and cause sliding to occur at lower load, thus reducing the compressive strength. Facaoara(146) suggested that the friction forces developed on the surface between large aggregate and mortar are stronger when the concrete is dry, while for concrete under water, the coefficient of friction is much lower.

Plain concrete(42) can be regarded as a chain comprising three links - cement paste, aggregate, and cement-aggregate bond. The effect of absorbed oil on the bond strength between coarse aggregate and cement can be seen by comparing the amount of reduction in the oil saturated hardened cement paste, mortar, and concrete. In the test results given in tables (4.2, 4.8),

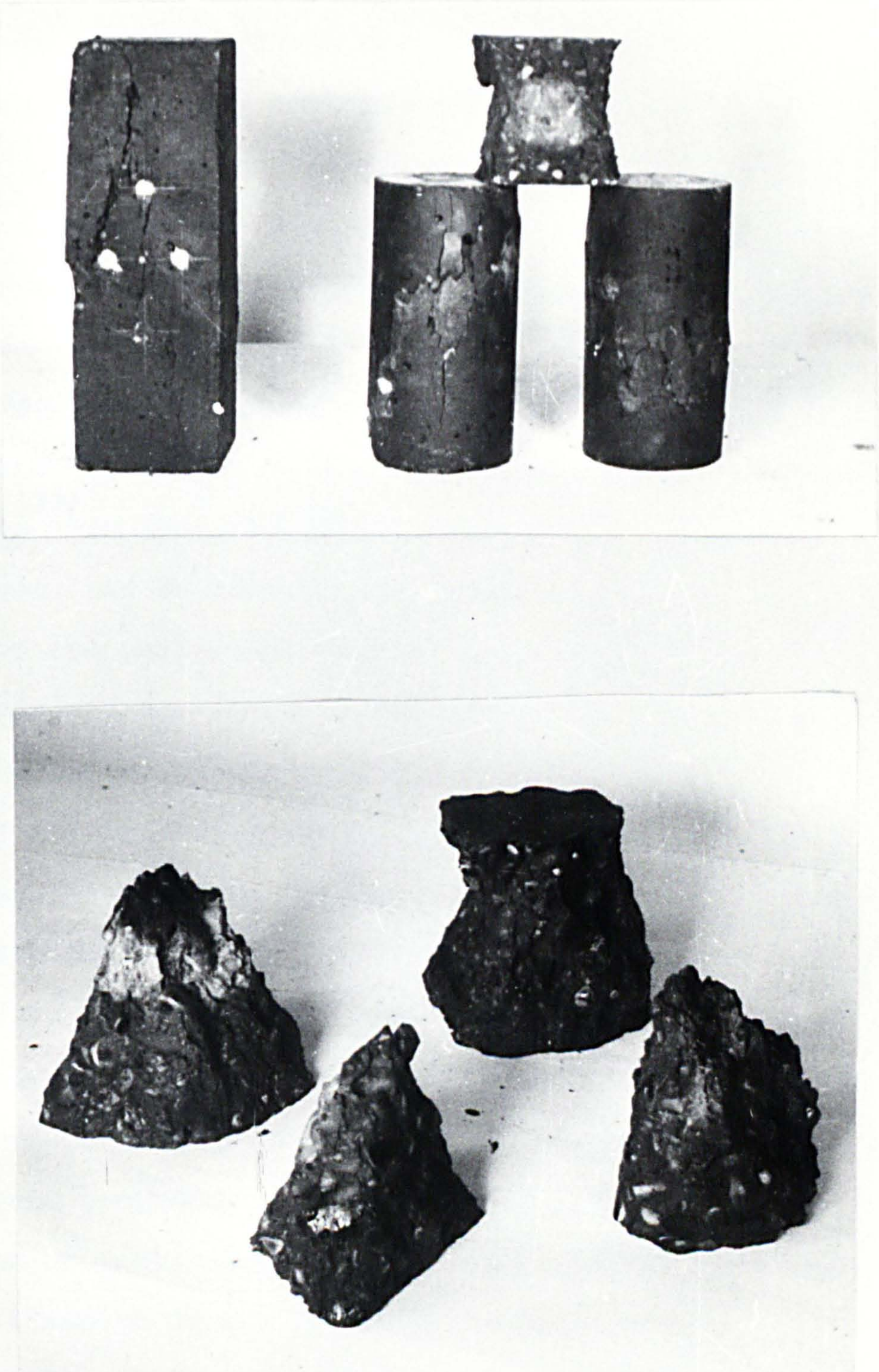


PLATE. 4-4. COMPRESSION TEST – TYPE OF FAILURE FOR CUBE, CYLINDER AND PRISM CONCRETE SPECIMENS

the reduction in strength of oil saturated specimens was between 4-8% for HCP,4.5-14% for mortar, 9.5-20% for 10 mm aggregate concrete, and 10.5-22.5% for 20 mm aggregate concrete as a percentage of oven dry strength before soaking in oil. Therefore, it is clear that the inclusion of coarse aggregate in the cement paste increases the percentage reduction of oil saturated specimens due to the effect of absorbed oil on the bond strength between coarse aggregate and cement paste.

The overall strength of concrete is also influenced by the size and topography of the coarse aggregate. Alexander and Wardlow(147) show that the bond increases with decreasing size of aggregate and decreasing w/c ratio. In fig. (4.54) which is a plot of the percentage decreases in strength for samples stored in crude oil for 600 days over the 28 day SSD (saturated surface dry) strength, against the total porosity, it is clear that a higher strength reduction due to absorbed oil was obtained in concrete with larger aggregate. The percentage reduction in strength of the sealed specimens was almost the same irrespective of the maximum size of aggregate used in the concrete mix. The presence of absorbed oil increased the reduction by 1.5-4.5% for the 10 mm aggregate concrete and by 2.5-6.5% for the 20 mm aggregate concrete. This is probably due to the fact that near the larger particles the cracks are long and may account for lower strength in the presence of oil.

4.5.3 The Effect of Absorbed Oil on the Cylinder and Prism Compressive Strength of Concrete.

It is well known(38) that due to the platen restraint effect, concrete cubes are loaded more nearly in a state of

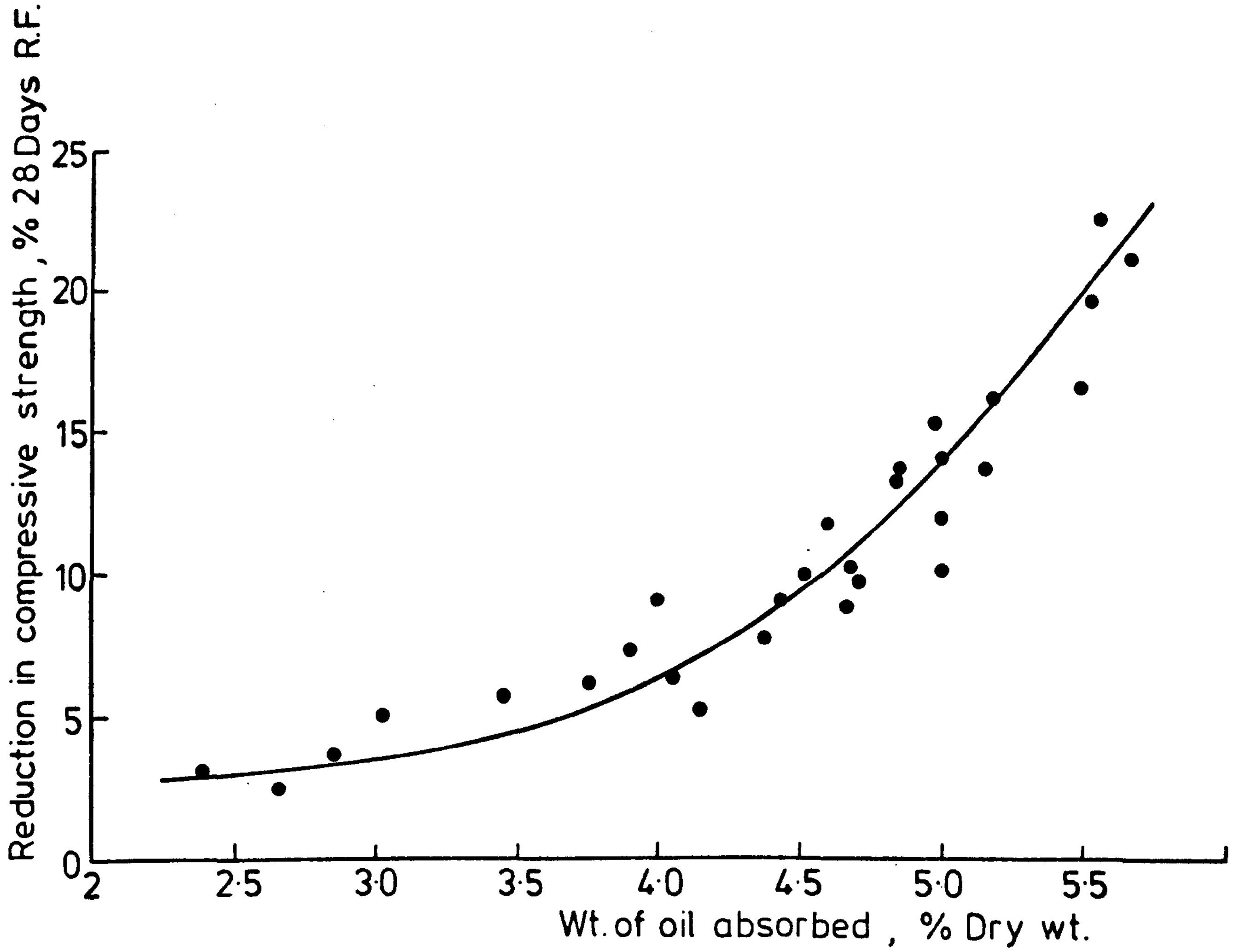


FIG. 4-53. PERCENTAGE REDUCTION IN STRENGTH DUE TO OIL SATURATION Vs. WT. OF ABSORBED OIL

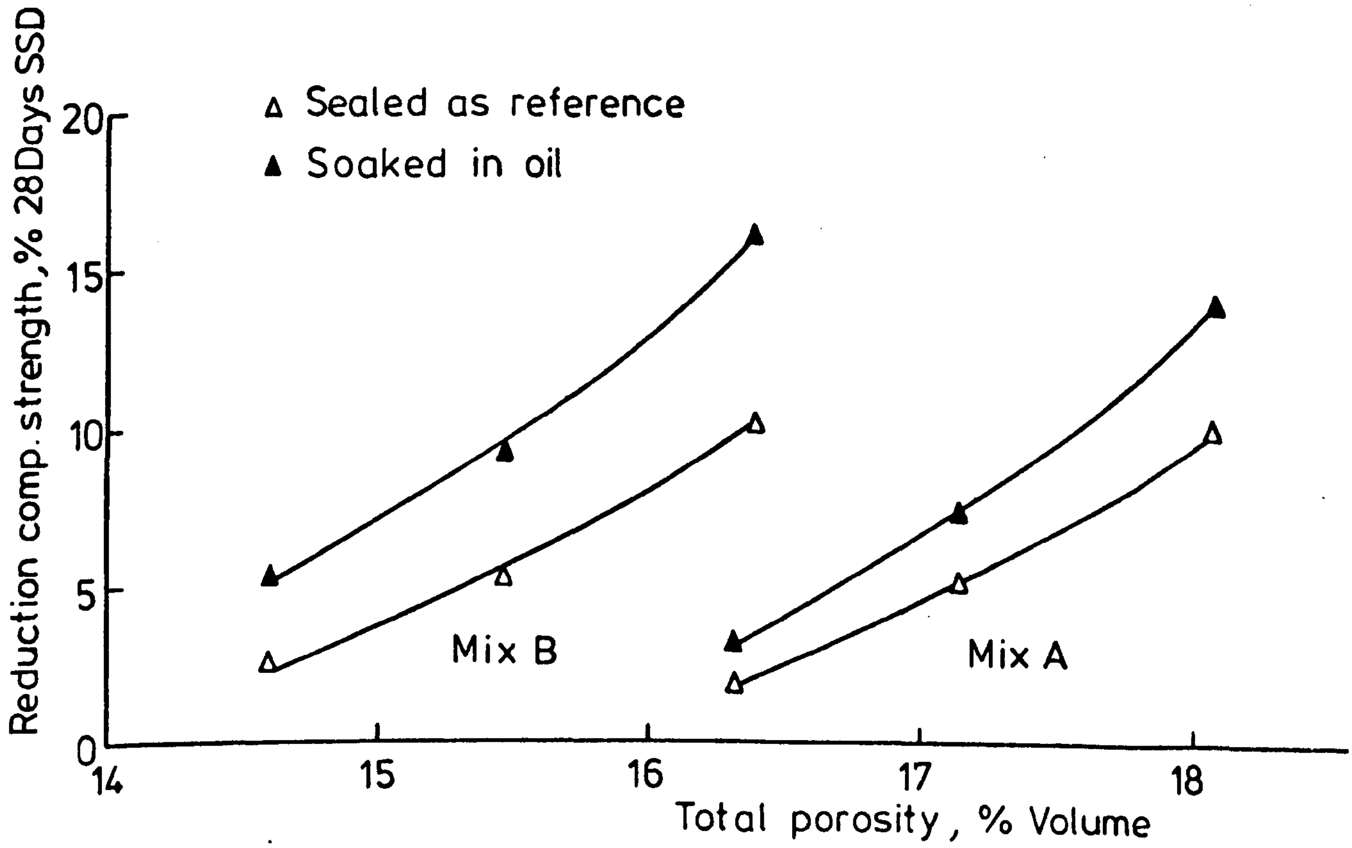


FIG. 4-54. REDUCTION IN STRENGTH OF CONCRETE DUE TO OIL SATURATION FOR 600 DAYS Vs. TOTAL POROSITY

triaxial compression while the cylinder, having a height to diameter ratio of two, fails more nearly in uniaxial compression. Therefore,to avoid the effect of the frictional restraint between the specimen and the platen of the testing machine and to obtain a true uniaxial compression failure, large specimens (cylinders of $\frac{h}{d} = 2$, prisms of $\frac{h}{w} = 3$) were used to study the effect of absorbed oil on the uniaxial compressive strength.

It was found in section (4.4.1.3) that the compressive strength of the oil soaked concrete, for both the cylinder and the prism, was decreasing with the increasing amount of absorbed oil, but the measured reduction was less than that for the sealed specimens. This might be due to the fact that the difference in strength of long and short specimens is related to their mode of failure. Cubes exhibited a shearing mode of failure on planes inclined to the direction of the applied load and a central part of the cube remained intact at failure. The presence of oil will decrease the coefficient of friction between the sliding planes as discussed in section (4.5.2). Long specimens exhibited vertical cracking parallel to the direction of the applied load and this extended throughout the cross section at failure.Thus, the tensile lateral stresses induced would cause a tensile mode of failure. The presence of oil might increase the tensile strength due to its viscose waxy characteristics. It was stated in chapter 3 that the analysis of the crude oil revealed wax content of 5.7% by weight. The deposited crude oil waxy molecules in the pore and cracks seem to increase the resistance against the induced tensile lateral stresses.

Some increase in the tensile strength due to the presence of oil might be caused by the closing and autogenous healing of the fine cracks in fractured concrete. This will be examined further in section (4.5.5).

As mentioned earlier, section (2.5.1), many investigators feel that shear failure of the cube specimens is not a true failure but a secondary effect caused by friction with the plates of the testing machine. The presence of oil between the end faces of the specimens and the platen of the machine would reduce the friction between them, and this would alter the stress distribution to a uniaxial stress and lower the load at ultimate failure. Examination of the type of failure of cubes showed that it was the same for all storage conditions and the effect of oil on the end faces was not significant. This effect is further investigated in figs. (4.55, 4.56) which are plots of the relationships between the cylinder-cube strength and the prism-cube strength respectively. The average cylinder-cube strength was 0.71 for the sealed specimens, 0.76 for oil saturated specimens, and .80 for water saturated specimens while the prism-cube strength ratio was 0.75, 0.77, and 0.79 respectively. The ratio for the oil saturated specimens was almost the same as for the water saturated specimens which is evidence of the minor effect of the oil on the end restraint of the testing machine.

4.5.4 The Effect of Oil Saturation on the Tensile Strength of Concrete.

In this investigation, the tensile splitting, flexural, and uniaxial tests were used to study the effect of absorbed

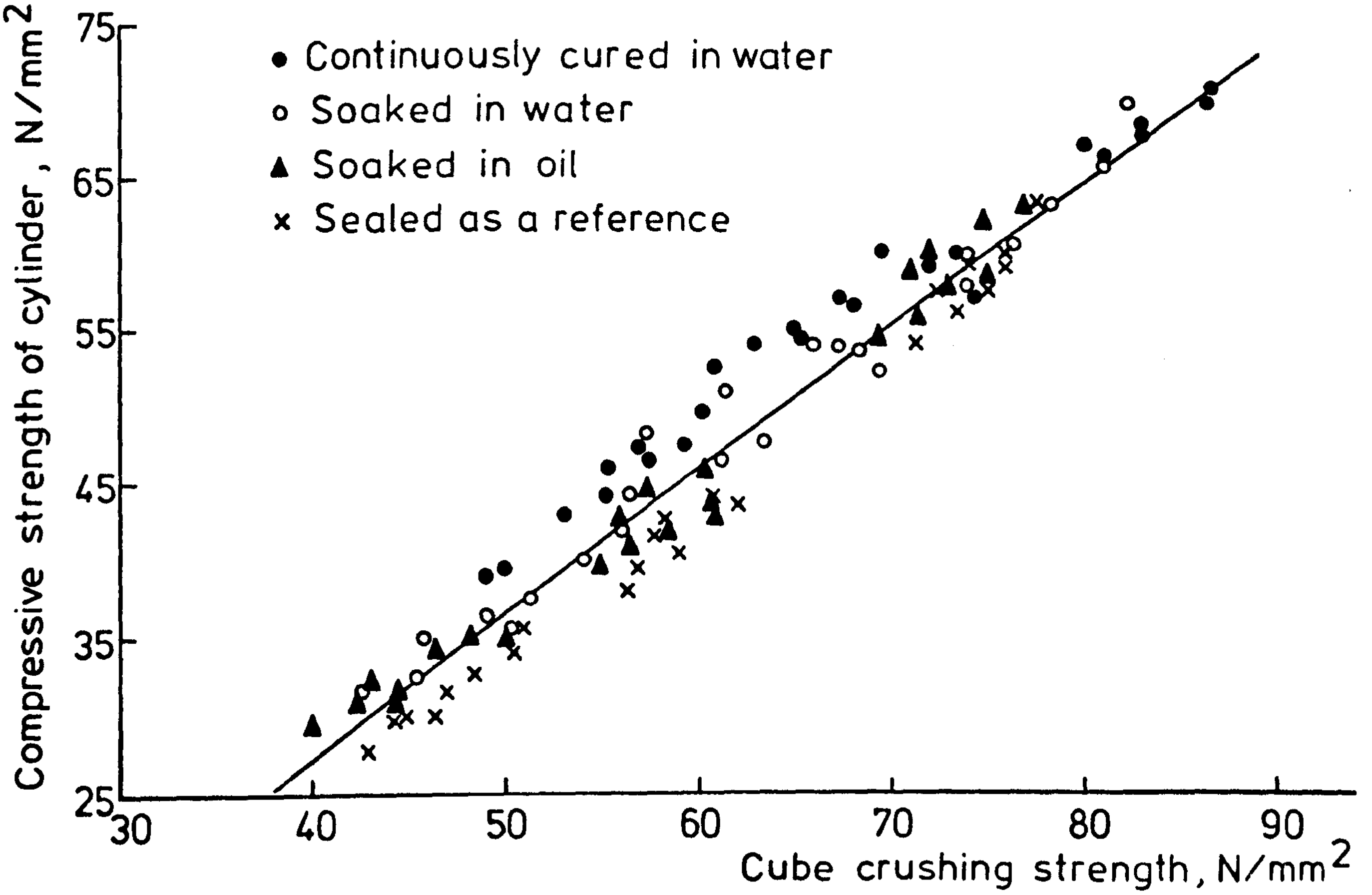


FIG. 4-55. RELATIONSHIP BETWEEN CUBE CRUSHING STRENGTH AND UNIAXIAL COMPRESSIVE STRENGTH OF CONCRETE CYLINDERS

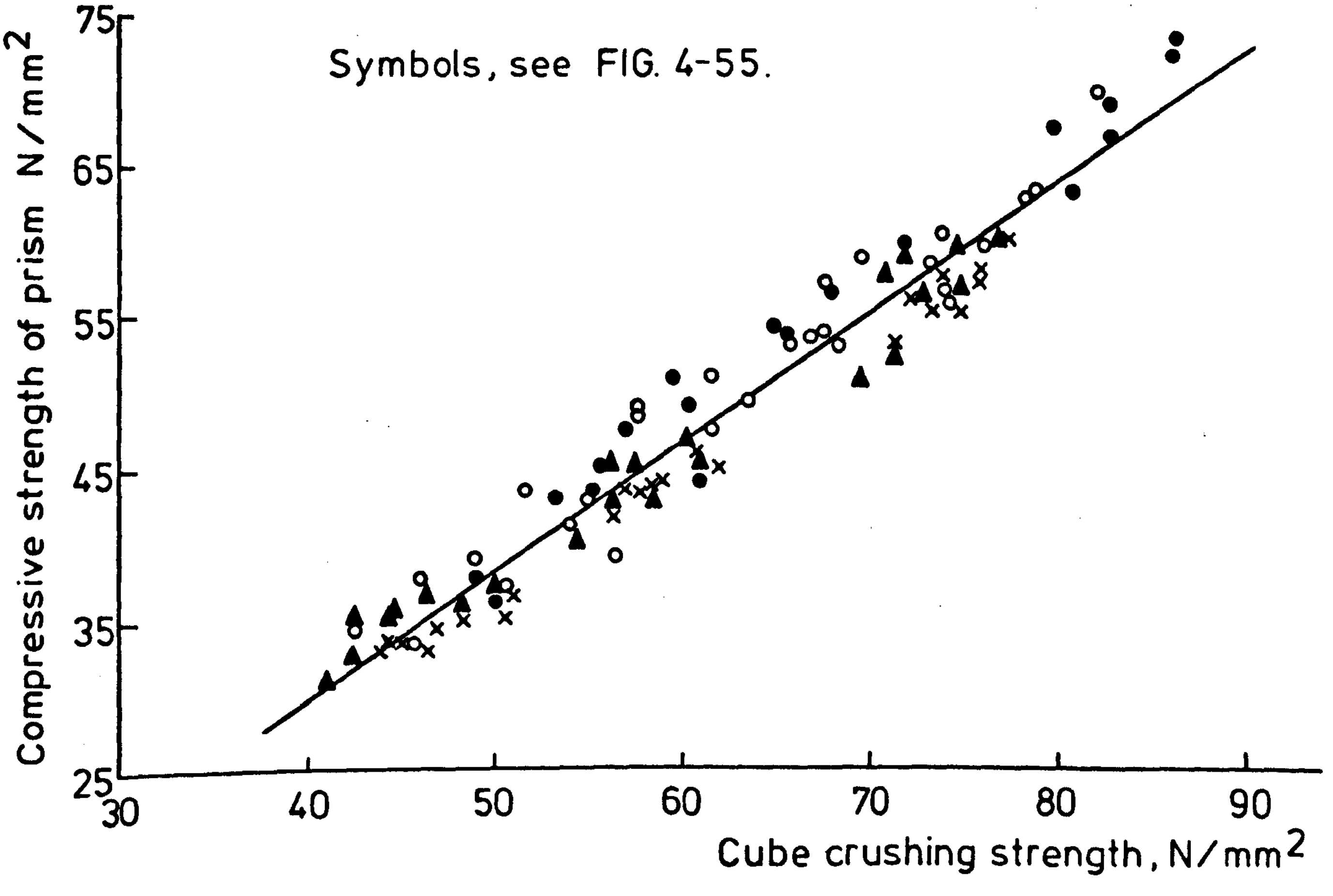


FIG. 4-56. RELATIONSHIP BETWEEN CUBE CRUSHING STRENGTH AND UNIAXIAL COMPRESSIVE STRENGTH OF CONCRETE PRISMS

oil on the tensile strength of concrete. The variable factors used in this investigation were the storage condition, the amount of absorbed oil (dependent on the soaking period), and the mix proportions.

The tensile strength when the dry specimens were soaked in oil, in contrast to soaking in water, showed an increase of about 3-6% for splitting tensile strength, 10-11% for uniaxial tensile strength, and 3-6% for modulus of rupture as a percentage of the dry strength within a soaking period of 24 hours. This sudden increase in the tensile strength might be related to the same reason which caused the immediate shrinkage (section 4.5.5) of the dry specimen as it soaked in oil. It is(148) known that the dry state of each gel particle is under a stress arising from its own surface tension. When this dry gel takes up water by adsorption, the physical reaction results in a release of energy in the form of heat, and the surface tension (interfacial tension) is diminished. After soaking in crude oil where immediate shrinkage takes place, the explanation might be reversed. The van der Waals forces increase as the gel particles come closer and produce an increase in the tensile strength.

The oven dried specimens kept continuously soaked in oil showed a continuous decrease with the increasing amount of absorbed oil. The highest reduction was between 7.5-9.5% and was almost the same for all types of tensile tests. This reduction might be due to the dilation of the cement paste and a weakening effect on the aggregate-cement bond.

The percentage reduction due to absorption of oil compared to that of the companion sealed specimens was almost the same for concretes of low w/c ratio but reduced as the w/c ratio increased, i.e. porosity increased and the amount of absorbed oil increased. This behaviour is somewhat similar to the uniaxial compressive tests on cylinders and prisms (Section 4.5.3). It has been found(124) that the mechanism producing the internal cracks in the uniaxial compression and in flexural tension is the same, and the cracks develop from local bond failure between the cement and the aggregate. Jones and Kaplan(37) found an excellent correlation between flexural strength and stress at which transverse pulse velocity began to decrease in the compression test. The conclusion drawn was that failure in the uniaxial compression and in flexural strength are essentially the same mechanism. The presence of oil would, therefore, affect the flexural strength in the same way as it affected the uniaxial compressive strength.

In figs. (4.57, 4.58), the flexural strength of concrete stored under different conditions was plotted against the cube crushing strength. The ratio of flexural strength to cube strength varied between 0.09-0.13 with an average of 0.105. This ratio decreased as the cube crushing strength increased. There was no single relationship for the concrete from different storage conditions and this might relate to the sensitivity of the flexural test to the moisture content since the flexural strength can be erratic when the moisture distribution(68) is non-uniform. Water soaked specimens showed a higher sensitivity and had a lower flexural/cube strength ratio compared to

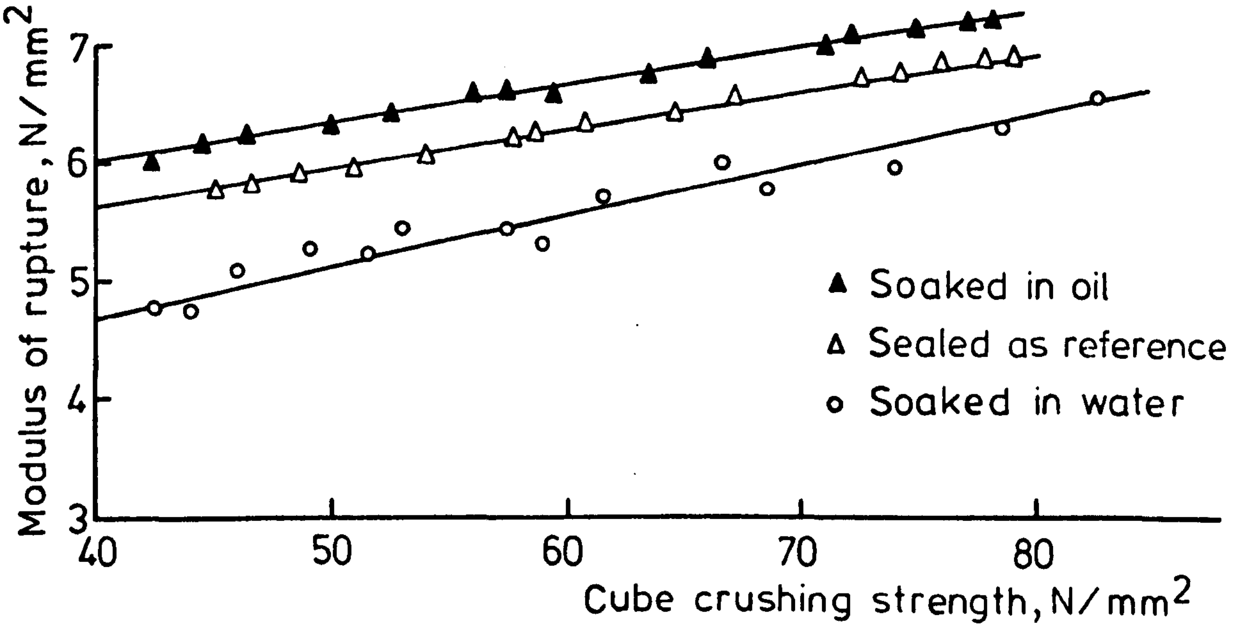


FIG. 4-57 RELATIONSHIP BETWEEN MODULUS OF RUPTURE AND CUBE CRUSHING STRENGTH OF CONCRETE, MIX A

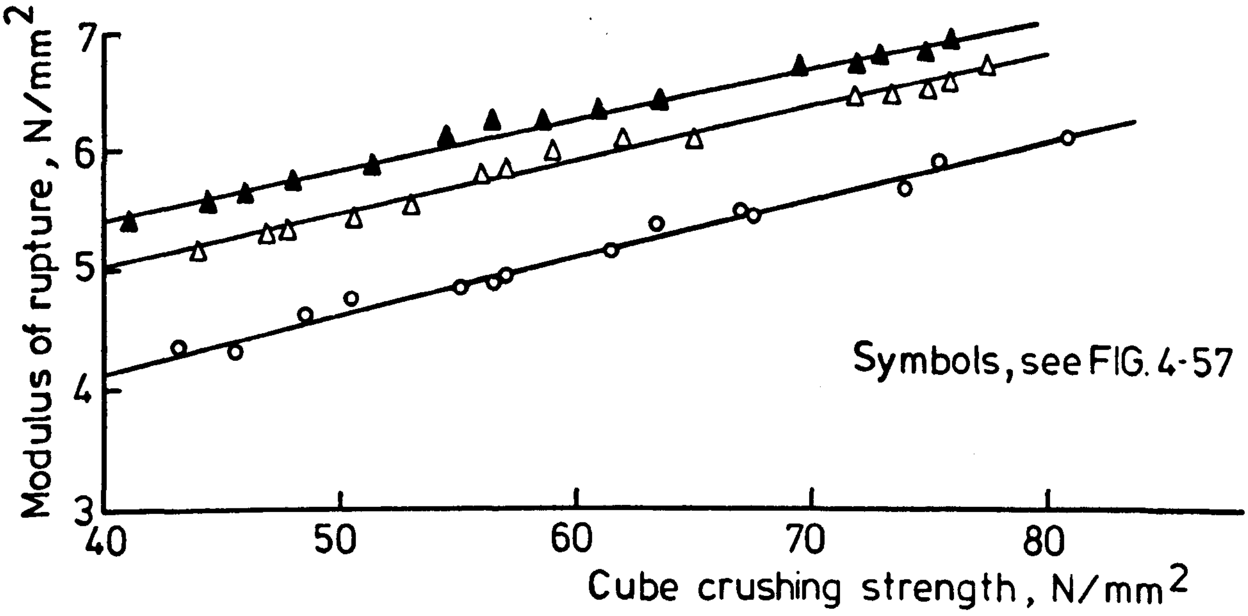


FIG. 4-58 RELATIONSHIP BETWEEN MODULUS OF RUPTURE AND CUBE CRUSHING STRENGTH OF CONCRETE, MIX B

that of oil saturated specimens. Very thorough drying followed by immersion in water caused sufficiently disruptive volume changes to reduce the strength.

Fig. (4.59) shows the relationship between the splitting tensile strength and the cube crushing strength of concrete dried and cured as described in Chapter (3). A single relationship, for the different curing conditions and for the different concrete mixes, can be obtained and expressed exponentially as

$$fsp = 0.15\ fcu^{0.78} \qquad \ldots\ldots \qquad (4.20)$$

The coefficient of correlation (r) equals 95.95. This single relationship suggests that, the non-uniform distribution of moisture in concrete does not effect the uniformity of either splitting tensile strengths or compressive strengths as much as it affects flexural strength. This might be related to the sensitivity of the outer fibres of the flexural specimen, which sustain the maximum stresses, to incidental moisture changes, while the maximum tension stresses in the split cylinder test are applied to the interior of the cylinder.

4.5.5 The Effect of Water and Oil Saturation on the Volume Changes.

Wetting the set cement paste results in the gel absorbing liquids and swelling. In the case of shrinkage, loss of water causes consolidation of the concrete. Removal of water brings the very large surface of the colloidal structure closer together and mobilizes strong surface forces. Only the gel water that is involved in the seepage movement has an effect on deformation, neither the capillary water nor the chemically combined water has any influence. The effect of water evaporating from the capillary pores is to produce tension which draws further water from the gel. Glocklich(149) states that the loss of free water causes little shrinkage but, subsequently, the removal of adsorbed water as well as interlayer water from the tobermorite phase is accompanied by a corresponding shrinkage in concrete.

The test results presented in section (4.4.9) showed that the dried specimens swelled immediately by various amounts depending on the cement and aggregate content, obviously, larger expansion was observed for concrete of higher cement content. This immediate swelling is caused by disjoining forces produced by adsorbed water in places where adsorption is hindered(148). This pressure, along with the change in surface tension of the solid material, accounts for the volume change by changing the distances between solid surfaces of the cement gel. It was also noticed that the dry specimens kept continuously cured in water continued to swell by various amounts as shown in figs. (4.35) to (4.38). The results are in agreement with other findings(70,71) where swelling is related to the continued hydration when a continuous supply of water is present(74).

The dry specimens soaked in crude oil showed an immediate shrinkage by various amounts depending on their mix design. The shrinkage continued but gradually slowed down until it reached its maximum value at about 100 days. Thereafter, the oil soaked specimens stopped shrinking and started swelling but the initial shrinkage was not recovered up to a soaking period of 600 days. The amount of volume changes in oil saturated specimens depends greatly on the amount of oil absorbed, pressure exerted in the pores of the cement gel, and its interaction with the cement gel particles. Crude oil is inert with cement gel and does not react with the remaining unhydrated cement particles, thus producing a weak mutual interaction and small volume changes.

The principal component of cement paste is the cement gel, which is an aggregation of colloidal particles in the form of slender ribbons and crumpled sheets or foils (148). The most important feature of the paste structure is the narrowness of many of the spaces that can be occupied by penetrating liquid. The estimated average surface to surface distance in the densest structure is about 15 Å and the minimum is perhaps 1 or 2 Å. Because of such proximity, the gel particles are held together by mutual attraction known as van der Waals forces acting across the very small distances between the cement gel particles. These forces are an important source of concrete strength and volume changes. The presence of water molecules in these interstitial spaces would exert a disjoining pressure acting against the van der Waals forces and thereby causing swelling of the cement paste. Less swelling is likely to be caused by crude oil, because oil molecules are large (large occurrence between 5-7 Å) and there is a considerable fraction of the pore space in cement paste which cannot be entered by them. These results are confirmed by (141,150) data showing that a considerable fraction of the pore space in cement paste cannot be entered by methanol molecules or other molecules larger than the water molecules. Powers (148) stated that there are some places in the structure of concrete paste which, whilst being accessible to water molecules are too small to receive benzene molecules. Hernikoff (143) found that steam-cured and oven dried neat cement paste would imbibe kerosene or light lubricating oil without swelling but subsequent soaking in water would lead to swelling without the prior removal of kerosene. It appears, therefore, that kerosene enters only the larger voids and is not adsorbed in the surface of the cement gel.

As already mentioned, the cement paste swells when a continuous supply of water is present, but when no moisture movement to or from the paste is permitted, autogenous shrinkage occurs. Crude oil absorbed acted as a sealing layer which prevents the exchange of moisture between the concrete and the ambient environment, and a resultant shrinkage occurred due to the loss of water held inside the concrete by a progressing hydration process.

From the test results presented in figs. (4.35) to (4.38), it can be seen that the amount of expansion and shrinkage strain reduced as the aggregate/cement ratio increased due to the restraining effect of the aggregate particles. Another noticeable result is that the expansion strain for water saturated specimens decreased as the maximum size of the coarse aggregate increased. The highest reduction was about 10% for concrete of 0.4 w/c ratio. Similar effects have been found by other investigators. Billing (151) stated that shrinkage is approximately proportional to the paste content per unit volume of concrete. Therefore, at a given grading, the volume change increases with the amount of cement paste. Washa (88) concluded that well-graded aggregates with a large maximum size of aggregates are effective in reducing shrinkage. He noted that concrete of the same cement content and slump containing ⅜ in. maximum size aggregate usually develops from 10 to 20% greater drying shrinkage than concrete containing ¾ in. maximum size aggregate, and from 20 to 35% greater than 1.5 in. maximum size aggregate concrete.

4.5.6 Effect of Oil Saturation on the Elastic Modulus of HCP, Mortar, and Concrete.

The test results for HCPs presented in section (4.3.3) showed that the elastic modulus of water saturated samples was higher than for the corresponding dry sealed or oil saturated samples. The elastic modulus was lowest for the oil saturated samples. Generally the factors which influence strength also influence the modulus of elasticity in a similar manner, though to a lesser degree. The reduction in elastic modulus found in crude oil saturation can be partially explained by the reasons given in section (4.5.2) for the compressive strength of hardened cement paste. The presence of oil might cause a weakening effect on the cement paste component due to the dilation of the cement paste and the reduction of the adhesive forces between the gel particles. Microcracking which could have occurred during drying and cooling processes and have caused the high amount of absorbed oil compared to mortar specimens, section (4.4.11), could also account for the reduction in the elastic modulus of oil saturated hardened cement paste. The presence of absorbed oil would reduce the frictional resistance between the sliding surface, thus reducing the elastic deformation under the uniaxial compressive load.

Cement paste has the lowest stiffness and showed a largely elastic behaviour and a brittle type of failure. The addition of sand increases the elastic modulus of the composite material, and the addition of coarse aggregate further increases the elastic modulus of the mortar.(152), the rate of increase being dependent on the volume content of rigid inclusions and the water content. The presence of rigid inclusions, on the other hand, imparts to the composite material a degree of inelastic deformation greater than that of the corresponding matrix. The presence of microcracks at the aggregate-matrix interfaces due to volume changes during setting and hardening, and the inherent heterogeneity due to the differential stiffness of the constituents caused the mortar and concrete to deviate from linearity at loads less than those for the corresponding matrix. The presence of absorbed oil in the pores and microcracks added another factor which effects the dimensional instability of mortar and concrete as shown in the test results presented in section (4.4.5).

The stress-strain relationships presented in figs. (4.7) and (4.24,25) for mortar and concrete respectively, show that at low loads the behaviour of the oil saturated specimens was similar to that of the sealed specimens. As the load inceased crude oil saturated specimens tended to be more flexible and weaker. The test results illustrated in figs. (4.24,4.25) showed that the onset of cracking for oil saturated specimens was detected at a stress (θ_i) lower than that for the water saturated specimens by about 14-22%. Using the ultrasonic technique, section (4.4.8), the onset of cracking for oil saturated specimens started at a stress which varies between 4-7% less than that for the sealed specimens and 18-35% less than for the water saturated specimens, depending on the volume of aggregate/cement ratio. It appears that apart from weakening the cement paste component, crude oil saturation also weakens the aggregate-cement bonds.

The test results presented in figs. (4.29,4.30) which are shown in a plot of applied stress against volumetric strain for concrete, showed that the critical stress (θ_{cr}) at which continuous crack patterns began to form was lower for oil saturated specimens than either of the sealed or water saturated specimens. The volume increase denoted by the descending part of the stress-volumetric strain curve was higher for oil saturated specimens than that of the other companion specimens and the peak of the stress-strain curve also decreased as a result of the reduction in the load carrying capacity of the cracked oil saturated concrete. The presence of oil seems to increase the deformation of the cracked surfaces by reducing the frictional resistance at the interfacial boundaries.

4.5.7 Relationship between Elastic Modulus and Compressive Strength of Concrete.

In general, it has been found that an increase in strength is associated with an increase (not at the same rate) in the modulus of elasticity of concrete. The British Code of Practice for the structural use of concrete CP110: 1972(153) relates the tangent modulus of elasticity E_c in KN/mm^2 to the cube strength (f_{cu}) in N/mm^2 by a square root expression,

$$E_c = 4.5 \sqrt{f_{cu}} \quad \ldots\ldots \quad (4.21)$$

Here the density of concrete is assumed as 2300 Kg/m^3. When the density of concrete is between 1400 and 2300 Kg/m^3, the British expression for the modulus of elasticity becomes:-

$$E_c = 0.8\ \rho^2\ \sqrt{fc} \times 10^{-6} \qquad \ldots\ldots \qquad (4.22)$$

The 1971 edition of the A.C.I. Building Codes gives E (psi) for normal weight concrete:

$$E_c = 57{,}000\ \sqrt{fcy} \qquad \ldots\ldots \qquad (4.23)$$

where fcy is the cylinder strength in psi.

Takabayashe(154) obtained a correlation for water cured specimens:

$$E_c = 8.0 \times fcy^{0.68} \times 10^4\ Kg/cm^2 \ \ldots\ldots \qquad (4.24)$$

and for air-cured specimens:

$$E_c = 7.7 \times fcy^{0.68} \times 10^4\ Kg/cm^2 \ \ldots\ldots \qquad (4.25)$$

In the present investigation, the experimental relationship between elastic modulus and compressive strength is illustrated in fig. (4.60). The relationship is not linear and can be expressed exponentially as:-

$$E_o = 2.8\ fc^{0.58} \qquad \ldots\ldots \qquad (4.26)$$

It is claimed that(153) the modulus values obtained according to expression (4.21) will be within ± 7% KN/mm^2 of the values obtained on loading tests in accordance with B.S. 1881, Part 5, 1972. Using the compressive strengths given in tables (4.7,4.8), the elastic modulus obtained from the experimental results - tables (4.13,4.15), are compared with the predicted values, using equation (4.21), in table (4.32). The difference varied between 2.6 and 6.8 KN/mm^2 depending on the strength of concrete and the storage condition of the test specimens.

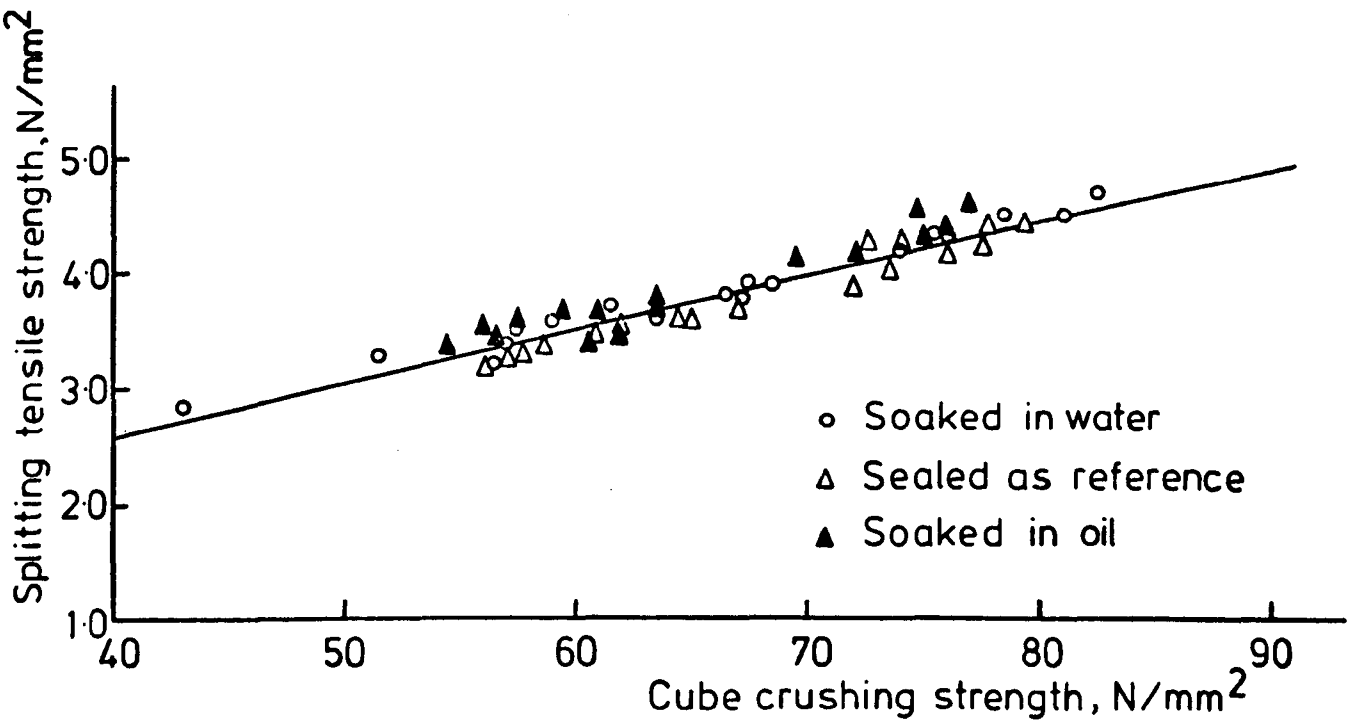


FIG. 4-59. RELATIONSHIP BETWEEN SPLITTING TENSILE STRENGTH AND CUBE CRUSHING STRENGTH OF CONCRETE

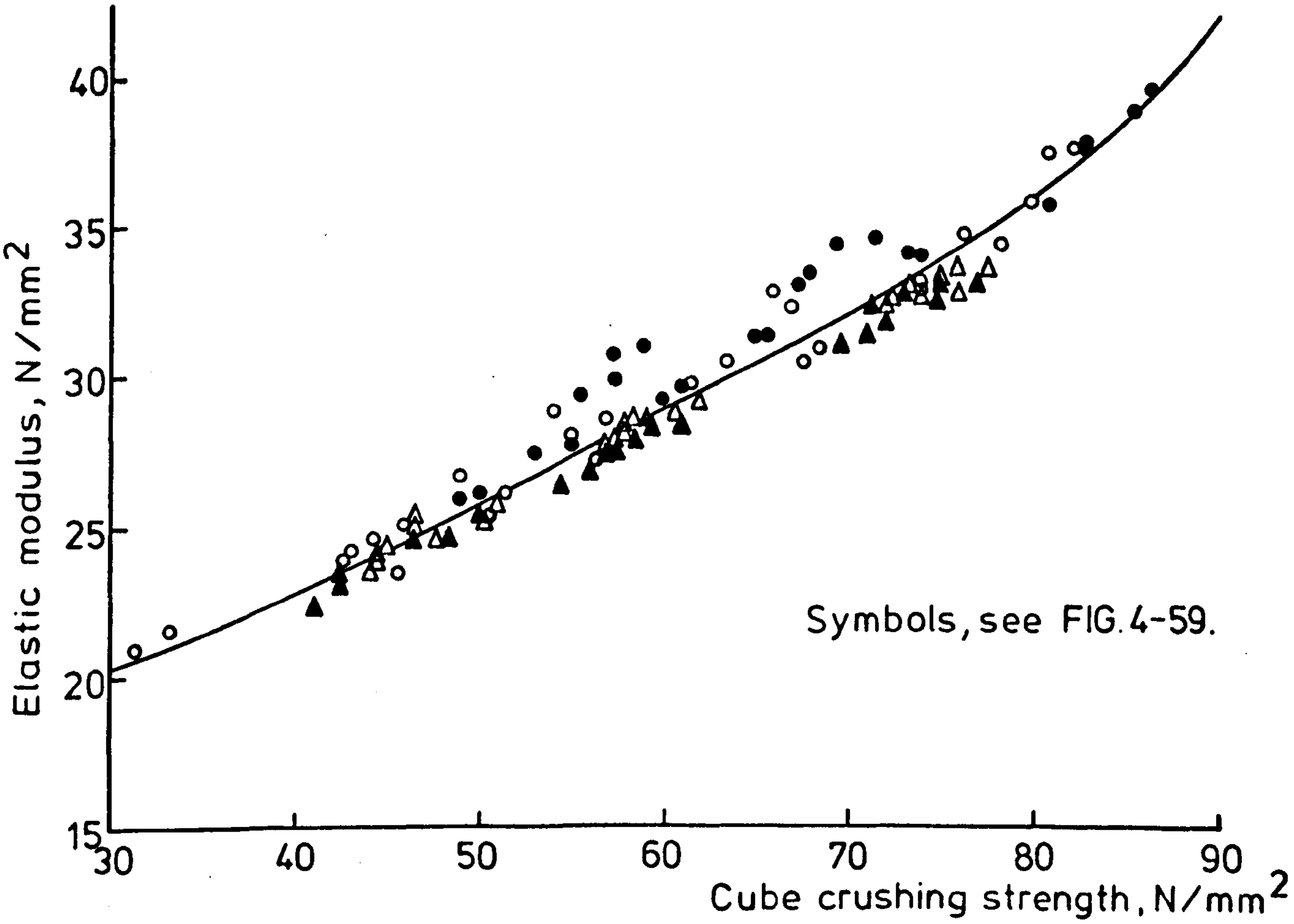


FIG. 4-60. VARIATION OF ELASTIC MODULUS WITH THE COMPRESSIVE STRENGTH OF CONCRETE

Specimens continuously cured in water			Dried specimens sealed as a reference			Dried specimens resoaked in water			Dried specimens resoaked in crude oil		
Code	Thesis	Difference	Code	Thesis	Difference	Code	Thesis	Difference	Code	Thesis	Difference
38.7	34.0	4.7	39.6	33.6	6.0	37.2	30.9	6.3	39.5	33.1	6.4
40.5	35.8	4.7	39.2	32.9	6.3	38.7	32.9	5.8	38.9	32.5	6.4
41.0	37.9	3.1	38.7	32.6	6.1	39.9	34.4	5.5	38.2	31.9	6.3
41.9	39.0	2.9	38.3	32.5	5.8	40.9	37.9	3.0	37.9	31.4	6.5
35.1	29.6	5.5	35.1	28.8	6.2	34.1	27.8	6.4	34.7	28.3	6.4
36.4	31.5	4.9	34.4	28.6	5.8	35.3	29.5	5.8	34.1	27.9	6.2
37.1	33.5	3.6	34.2	28.1	6.1	36.6	32.9	3.6	33.7	26.9	6.8
38.0	34.6	3.5	32.1	25.8	6.3	29.4	23.9	5.5	31.8	25.4	6.4
31.8	26.1	5.7	31.3	25.4	5.9	30.5	25.1	5.4	30.7	24.6	6.1
33.4	27.8	5.6	30.7	25.0	5.7	31.5	26.8	4.7	30.0	24.1	5.9
34.1	29.9	4.2	30.2	24.4	5.8	33.1	28.9	4.2	29.3	23.1	6.2
34.6	31.0	3.6	39.2	33.6	5.6	36.9	30.4	6.6	39.0	33.4	5.6
38.6	34.1	4.5	39.0	33.4	5.6	38.7	33.0	5.7	38.4	32.8	5.6
40.2	35.9	4.3	38.6	33.0	5.6	39.4	34.8	4.6	38.1	32.4	5.7
41.0	37.6	3.4	38.2	32.4	5.8	40.5	37.5	3.0	37.5	31.1	6.4
41.6	39.0	2.6	35.4	29.1	6.3	33.8	27.3	6.5	35.1	28.6	6.5
34.8	29.3	5.6	34.6	28.5	6.1	35.3	29.6	5.7	34.4	27.9	6.5
36.2	31.3	4.9	34.3	28.4	5.9	35.9	30.5	5.4	32.8	27.4	6.4
36.9	33.1	3.9	34.0	27.6	6.3	36.8	32.4	4.4	33.2	26.4	6.4
37.5	34.3	3.2	32.0	25.1	6.8	30.4	23.5	6.8	31.2	24.6	6.6
31.5	26.0	5.5	31.0	24.6	6.5	31.9	25.4	6.6	30.0	23.9	6.1
32.8	27.5	5.3	30.0	24.1	5.9	32.3	26.1	6.2	29.3	23.3	6.0

Table(4.32) Comparison of the experimentally obtained Elastic Modulus with that predicted from CP110.

CHAPTER 5.

BOND CHARACTERISTICS OF OIL SATURATED CONCRETE.

5.1 INTRODUCTION.

The present work deals with the behaviour and the distribution of anchorage bond of oil saturated specimens compared to those resoaked in water or those sealed dry as control specimens. Tests were carried out at different soaking periods to study the variation in the bond strength with the amount of absorbed oil.

5.2 SCOPE OF THE TESTS.

The present work can be divided into two parts as follows:

1. A concentric cylindrical pull-out test with specimens reinforced with only one central bar, deformed or plain, and the pull out load and slip of the free end of the bar relative to the mass of the concrete were measured.
2. A modified pull-out test was used to measure the bond stress along a plain bar centrally embedded in concrete using an electrical resistance gauge installed inside the bar, see section (5.3.2).

The purpose of these two sets of experiments was to compare the following results from oil saturated, water saturated, and dry sealed specimens.

a. Average bond strength.

b. Load transfer and bond stress distribution.

c. End slips.

5.3 EXPERIMENTAL PROGRAMME AND TEST PROCEDURE.

5.3.1 Pull-Out Test.

5.3.1.1 Steel Reinforcement.

High stress - high bond deformed bars and plain round mild steel bars, both of 20 mm diameter were used in the tests.

The minimum yield point was 340 N/mm^2 for plain bar and 506 N/mm^2 for deformed bar. The surface deformation was according to the requirements of B.S.4449.(155)

To evaluate the average modulus of the reinforcing bar, the elongation of the bar over a 50 mm length was measured using an extensometer placed at the central portion of the tested bar. Two specimens were tested, and gave similar results. The extensometer readings for the two specimens at each loading stage were averaged, and the curves shown in fig. (3.3) were plotted. The elastic modulus was 235 kN/mm^2 for the plain round mild bar and 255 kN/mm^2 for the deformed bar.

5.3.1.2 Test Programme.

The investigation covers two concrete mix designs, (A) using w/c ratio = 0.4 and, (B) using w/c ratio = 0.6, and all other factors which effect the preparation of the concrete mix were kept constant as far as possible. The details of the mix proportions are given in table (5.1). For each concrete mix and type of reinforcing bar, nine specimens were cast and water cured for 7 days. The specimens were then dried in an oven at 105° C for sufficient time to reach an equilibrium weight. After the drying process, the nine specimens were divided equally into three groups; the first were soaked in crude oil, the second in water, and the third were sealed in polythene bags and stored as control specimens. The details of the curing condition are described in Chapter 3. The pull-out specimens were tested at 120, 300, and 750 days to study the effect of different degrees of oil saturation on the bond strength.

5.3.1.3 Specimen Preparation.

The specimens were cylindrical, 150 mm in diameter and 150 mm in height. They were laterally reinforced with a helix bar

6 mm diameter of mild steel with a pitch of 25.4 mm to prevent splitting or bursting failure in the concrete cylinders.

The specimens were cast in steel mould cylinders measuring (150x300 mm) as shown in plate (5.1). The cylinder length was reduced to 150 mm by using special forms. The reinforcing bar was supported vertically by clamps to coincide with the central axis of the cylinder. It projected about 10 mm from the embedded face and about 450 mm from the other face (loaded end). Before casting, the end of the bar projecting 10 mm was ground smooth, whilst the 450 mm length was cleaned with a steel wire brush to remove the rust, and wiped with acetone to remove any oils. The detail of casting is described in chapter 3, section (3.5).

Max. size of aggregate mm	Mix Designation	w/c ratio	Cement Content Kg/m^3	Proportion by weight		
				Cement	Sand	Gravel
10	A	0.4	575	1	1.22	1.50
	B	0.6	383	1	2.24	2.44

Table (5.1) Details of concrete mixes used in the bond tests.

5.3.1.4 Testing Procedure.

An Avery universal testing machine of 1000 kN capacity, was used to apply the pull-out force at a constant rate of 13-15 N/mm^2. The specimen was placed on a special rig as shown in fig. (5.1). The specimen was seated on a system of bearing plates and a spherical bearing block to ensure that the load would be purely axial. A plywood pad 3 mm thick was inserted between the concrete specimen and the steel bearing plate. The

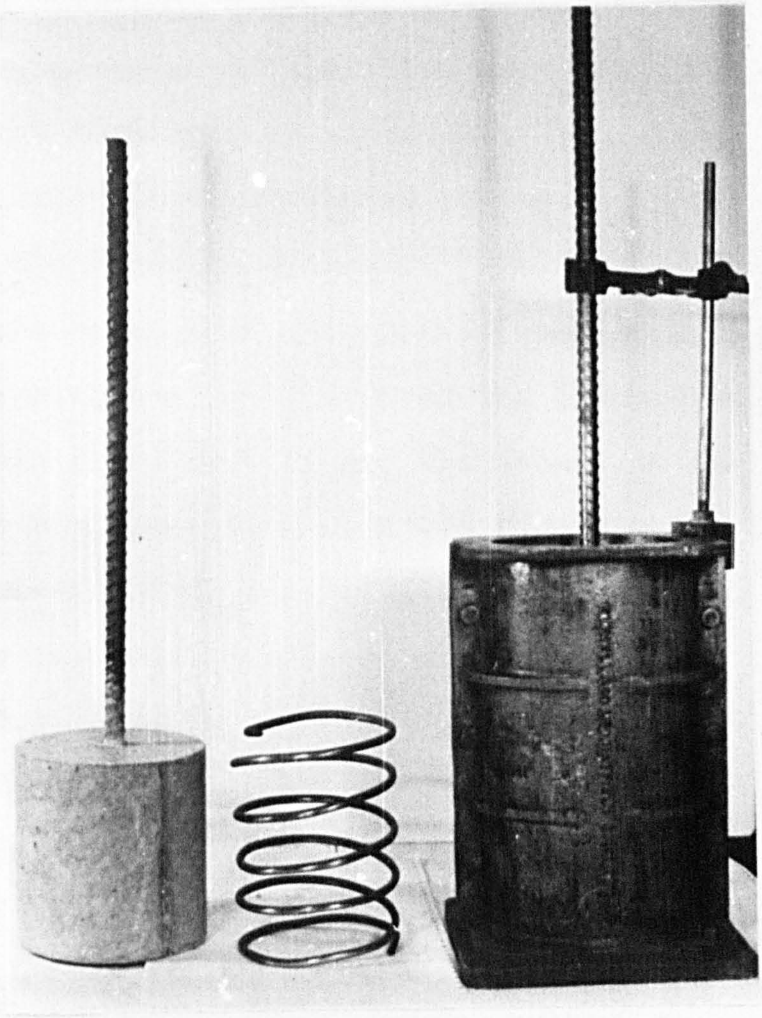


PLATE. 5-1. DETAIL OF THE PULL-OUT TEST SPECIMENS

dial gauge used to measure the free end slip was mounted on a support that was attached to the top face of the concrete by bolts secured to the side of the specimen with the point of the dial resting on the planed end of the bar.

All specimens taken out of water or crude oil tanks were wiped clean and immediately tested. The load and slip of the bar at the free end was recorded. The loading continued until the ultimate load was reached and then the bar was pulled through the concrete cylinder. The maximum load necessary to detach the steel from the concrete was recorded and the average bond strength was calculated on the assumption that all bars were plain round bars. The withdrawal of the bar from the concrete block was also obtained from a graph drawn automatically by the testing machine during the experiments. Graphs of this type are included in section (5.4.1.1).

5.3.2 Modified Pull-Out Test.

5.3.2.1 Test Programme.

Concrete mix designs, curing, and storage conditions were the same as described in section (5.3.1.2). The specimens were tested after a soaking period of 8 months to ensure a high degree of saturation with crude oil. The main reinforcing bar was a plain mild steel bar having a diameter of 18 mm., an ultimate strength of 553 N/mm^2, and a modulus of elasticity equal to 231 kN/mm^2. The deformed bar was used to ensure that slip failure of the specimen would occur by pulling-out along the instrumented plain bar only, fig. (5.2).

5.3.2.2 Instrumented Bar.

Load distribution along the bar was obtained using internal electric resistance strain gauges. Each instrumented bar was

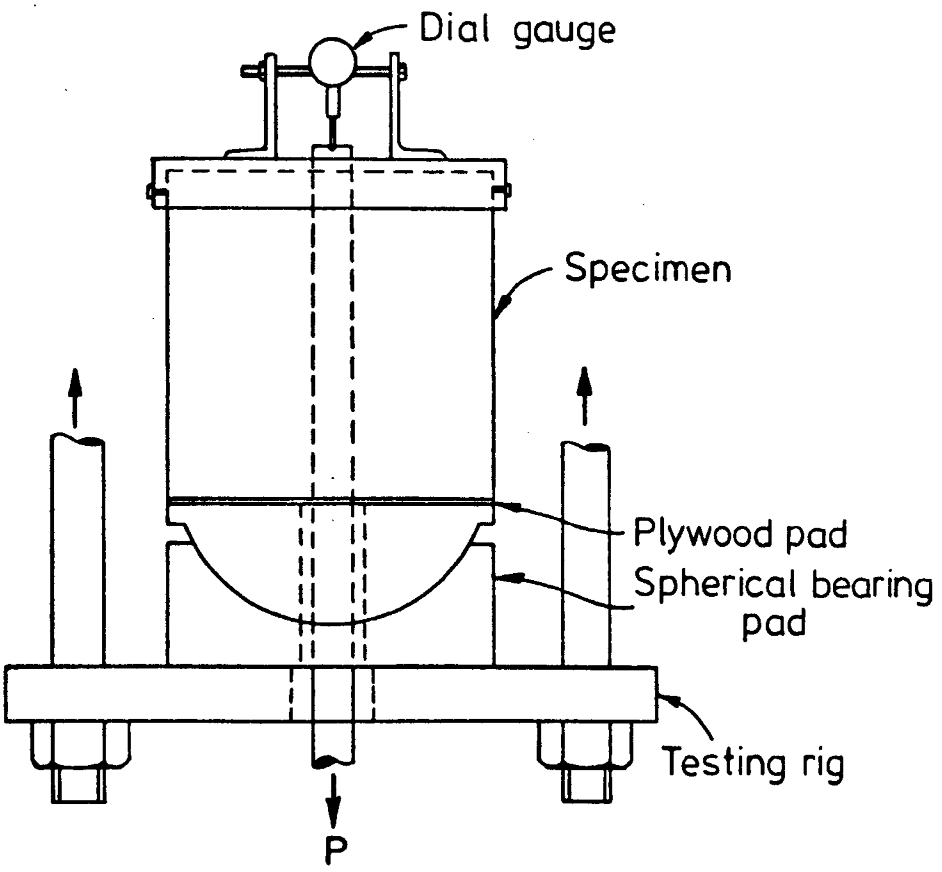
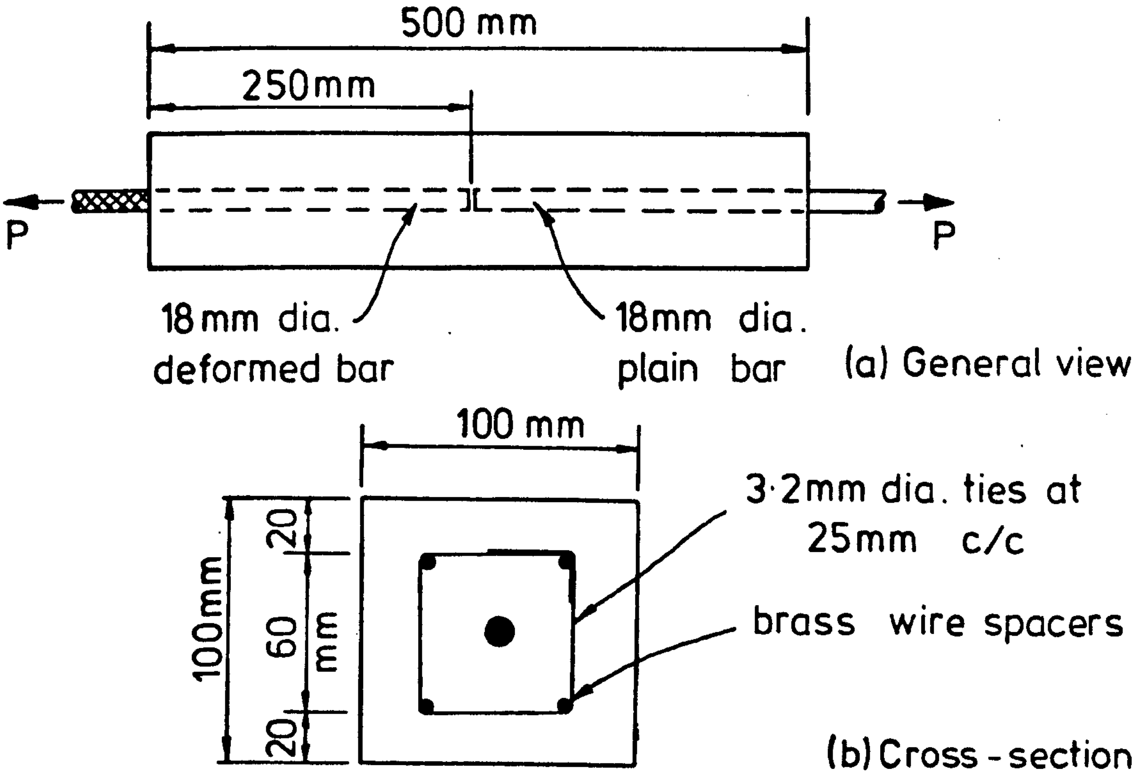


FIG. 5-1. DETAILS OF PULL-OUT TESTING ARRANGEMENT



(a) General view

(b) Cross-section

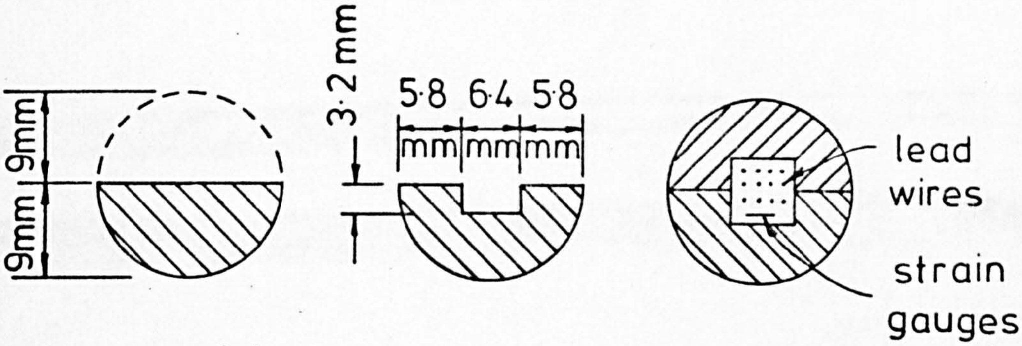
FIG. 5-2 DETAILS OF THE MODIFIED PULL-OUT SPECIMEN made from two round steel bars 18 mm in diameter milled to semicircular section with a channel of 6.35 mm wide and 3.18 mm deep milled out of the centre of the flat surface of the two half bars. The 6 mm TML (QFLA-6) 120 ohm gauges were bonded to the surface of this channel as shown in fig. (5.3) and plate (5.2). The gauges were installed in the base of the channel, spaced at 51 mm, and glued with rapid (P-2) adhesive and wired using two No. 24, plastic insulated, stranded copper wires which were running in the channels of both bar halves to the outlet at the end of the bar.

Following the insulation and checking of the gauges, two coats of a water resistant silicone resin was applied to all gauges and connections. The channels were then filled with epoxy resin. After hardening, a new layer of epoxy was applied to fill out the remaining depressions and cover the contact surface of each half-bar. They were then clamped together and tack welded at 50 mm centres using a welding sequence, with intermittent cooling to protect the gauges from damage. Five inches of continuous weld was used at each end of the bar gripped by the tensile machine. The gauges were not damaged by the welding temperatures.

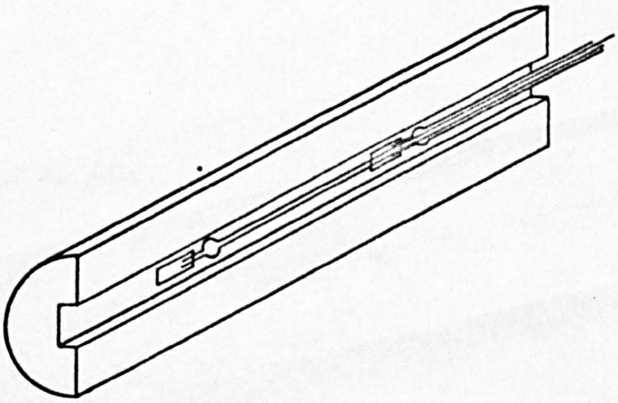
Finally, the bars were carefully cleaned with a wire brush and wiped with acetone before the casting of concrete. The bars so formed had an exterior appearance almost identical to that of a standard solid bar.

5.3.2.3 Preparation of test Specimens.

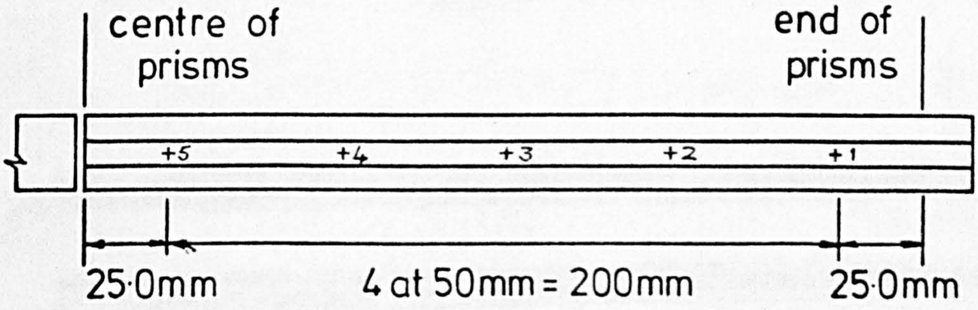
The specimen shown in fig. (5.2), was cast in steel mould prisms (100x100x500 mm.). Transverse reinforcement consisted of 3.18 mm. diameter mild steel bar ties attached to four thin



(a) Steps of bar preparation



(b) Strain gauges mounted in groove



(c) Location of strain gauges

FIG. 5-3 STEPS OF BAR PREPARATION AND GAUGE LOCATIONS FOR THE MODIFIED PULL-OUT TEST

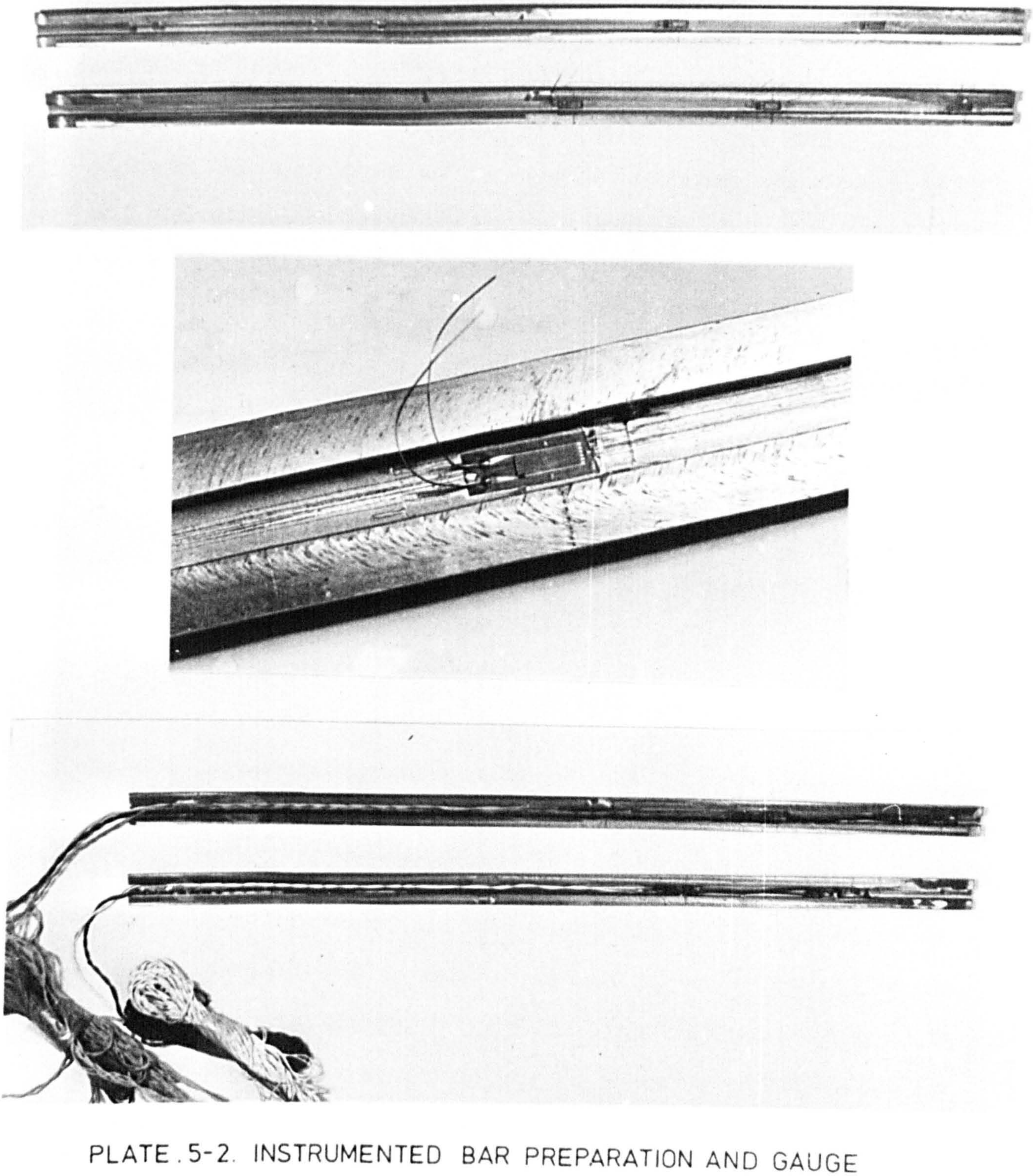


PLATE. 5-2. INSTRUMENTED BAR PREPARATION AND GAUGE LOCATION FOR MODIFIED PULL-OUT TEST

brass wires at 25 mm. pitch placed inside the mould as shown in fig. (5.2). The brass wires had no appreciable effect in the longitudinal direction.

Two circular holes were drilled at the ends of the steel moulds to accommodate the protruding ends of the two main reinforcing bars, the deformed and the instrumented bars, each protrusion measuring 200 mm. This length was adequate for placing the specimens in the pressure vessel containing the crude oil and also for gripping by the jaws of the testing machine. The instrumented bar and the deformed bar, for each specimen, were placed horizontally through the hole of the mould. They were aligned centrally along the prismatic axis and they were fixed firmly by special fittings at the ends of the mould. This attachment helped to hold the bars against any displacement caused by the vibration applied to the specimen during the casting operation.

5.3.2.4 <u>Testing Procedure.</u>

All specimens were tested in a universal Avery testing machine as shown in plate (5.3). For all tests, the load was applied incrementally from zero to the limit of validity of the instrument readings (generally established by the tensile failure of the specimen at the centre), at intervals of 5 kN. At each load increment all the steel strain measurements were recorded manually using the automatic selector type 1542 and the strain gauge apparatus Type 1516 as shown in plate (5.3). The slip of the instrumented bar in relation to the mass of concrete was measured by a 0.01 mm. micrometer dial gauge. The dial gauge was mounted on clamps fixed to the instrumented bar with set screws; the slip was measured at 10 mm. from the

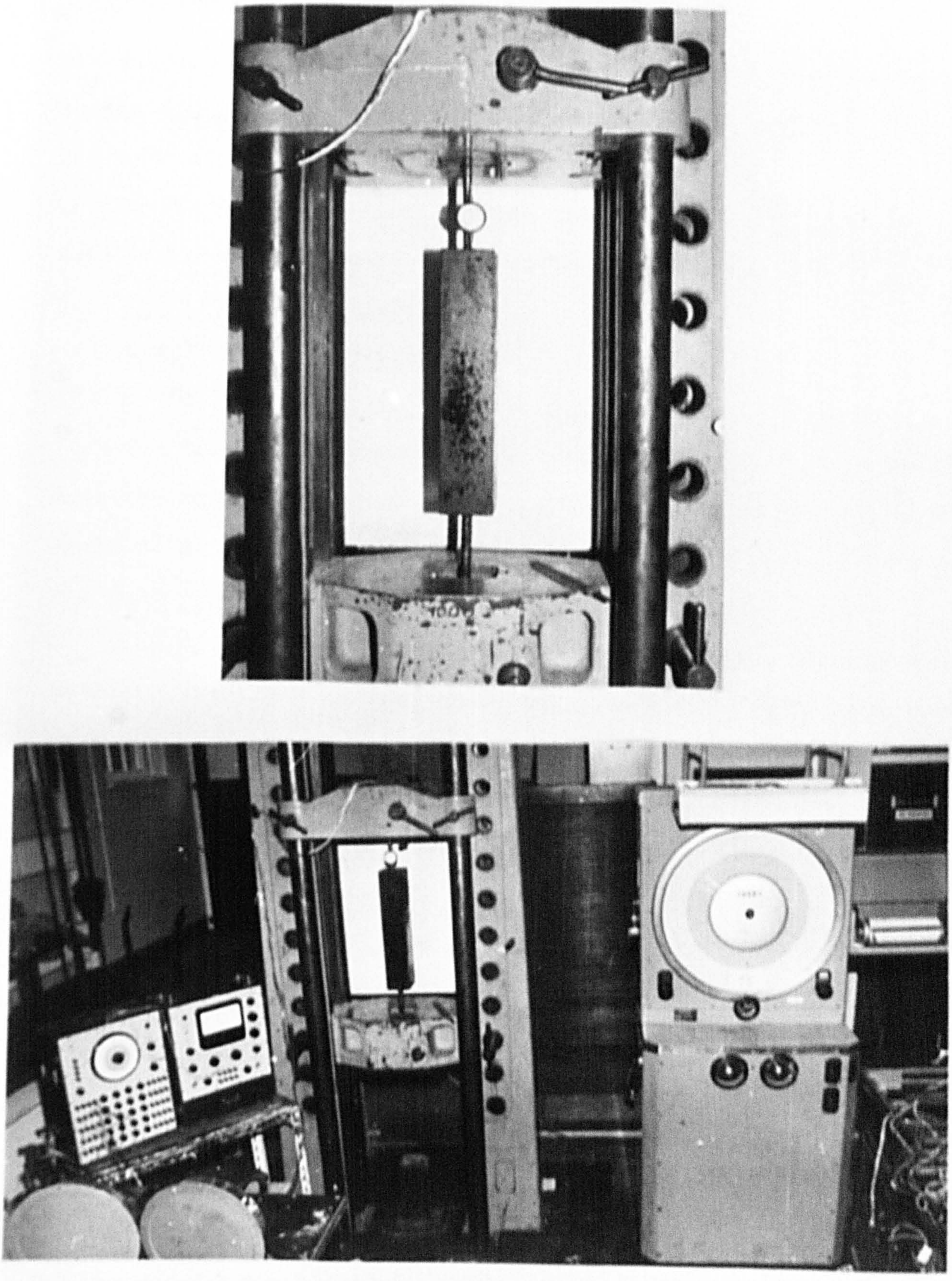


PLATE 5-3. MODIFIED PULL-OUT TEST USING AVERY UNIVERSAL TESTING MACHINE

bar surface using a lever system. Slips measured in this manner are a close representation of the actual relative displacement of the bar from the surrounding concrete. The readings of the strain gauges and the dial gauge were recorded after each load increment and loading was continued up to failure.

5.4 TEST RESULTS AND DISCUSSION.

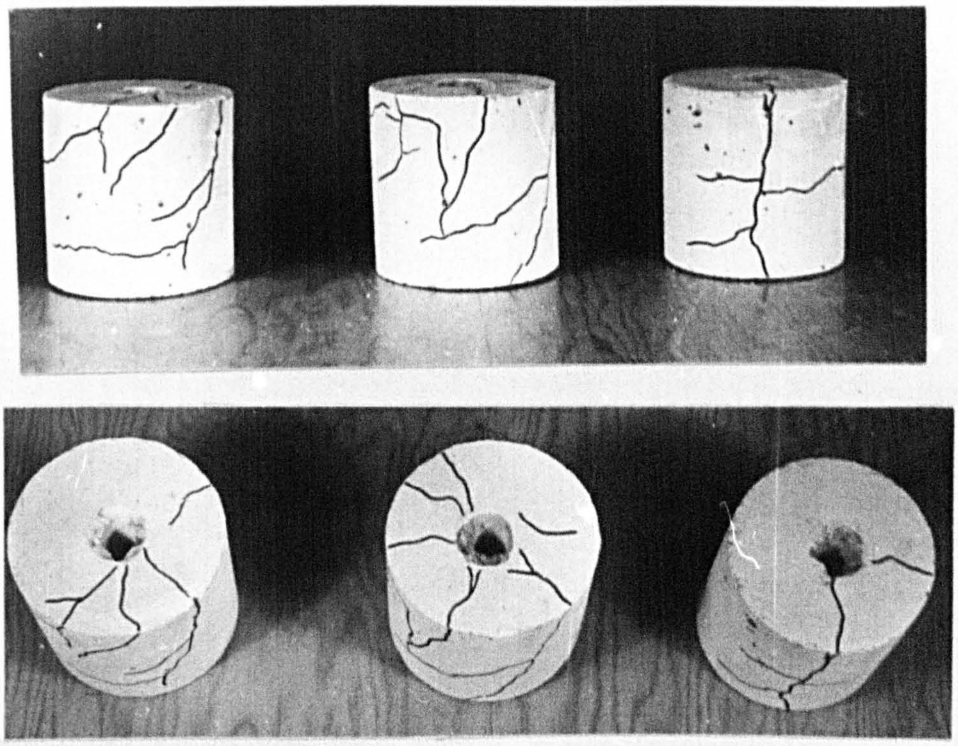
5.4.1 Pull-Out Test.

5.4.1.1 Behaviour at Failure.

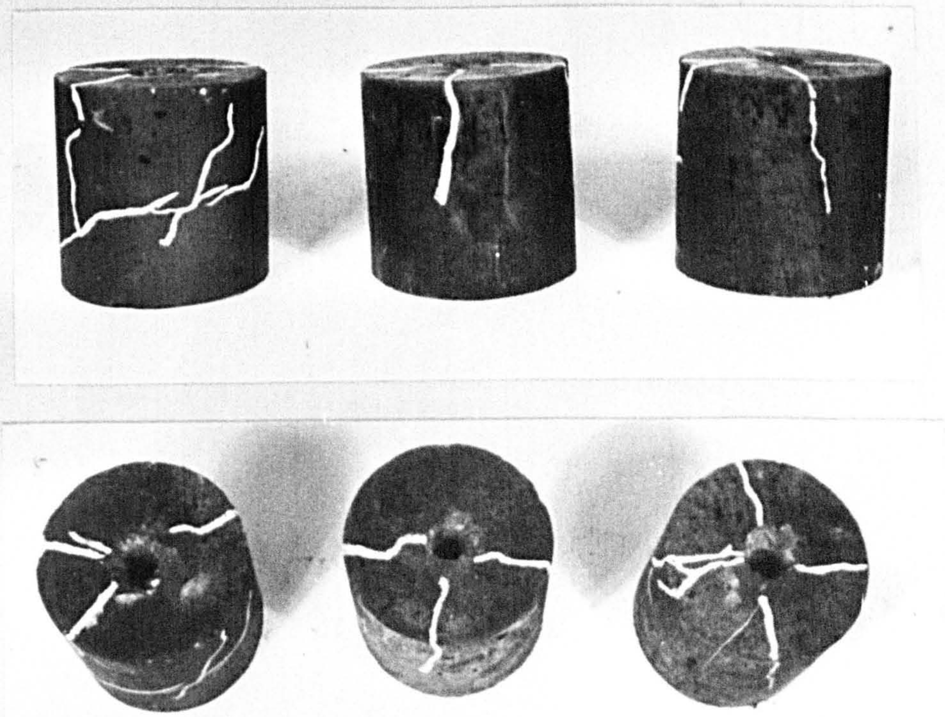
The specimens were loaded in increments which were decreased near the expected maximum load to give time for more slip measurements to be taken. After failure the load decreased as the bar was pulled out and slip measurement became impossible.

The plain bar was pulled out without cracking the concrete in every case, but the deformed bar always cracked the concrete in spite of having traverse reinforcement. The cracks started over the bar at the loaded end and gradually progressed over the entire length before pull out was complete. As the bar was pulled out of the concrete, some crushed concrete was found between the bar lugs. The crack patterns for the concrete specimens of the deformed bar, stored at different conditions, are shown in plate (5.4).

Figs. (5.4, 5.5) show that the displacement of the plain bar increased slowly at first then more rapidly, until finally the load suddenly dropped as the bar was pulled out. For the deformed bar, the load did not decrease suddenly, but remained approximately constant over a relatively large range of displacement and then reduced gradually. In the case of oil saturated specimens, for both plain and deformed bars, the load-slip relationship beyond the ultimate load started to fluctuate as



(A) SOAKED IN WATER



(B) SOAKED IN OIL

PLATE .5-4. CRACK PATTERNS OF PULL-OUT TEST USING DEFORMED BAR

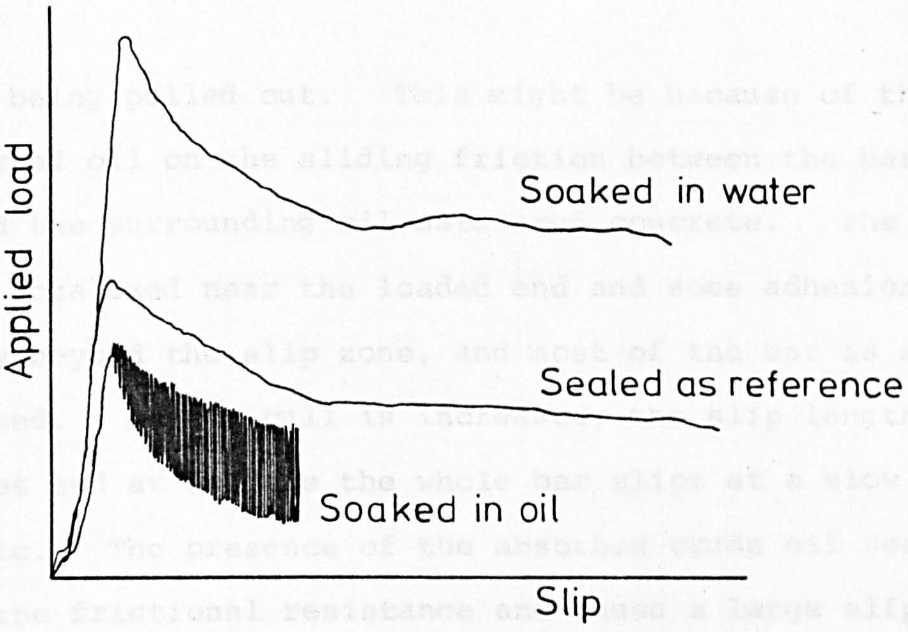


FIG.5-4 TYPICAL LOAD-SLIP CURVES AS DRAWN AUTOMATICALLY BY THE TESTING MACHINE - PLAIN BARS

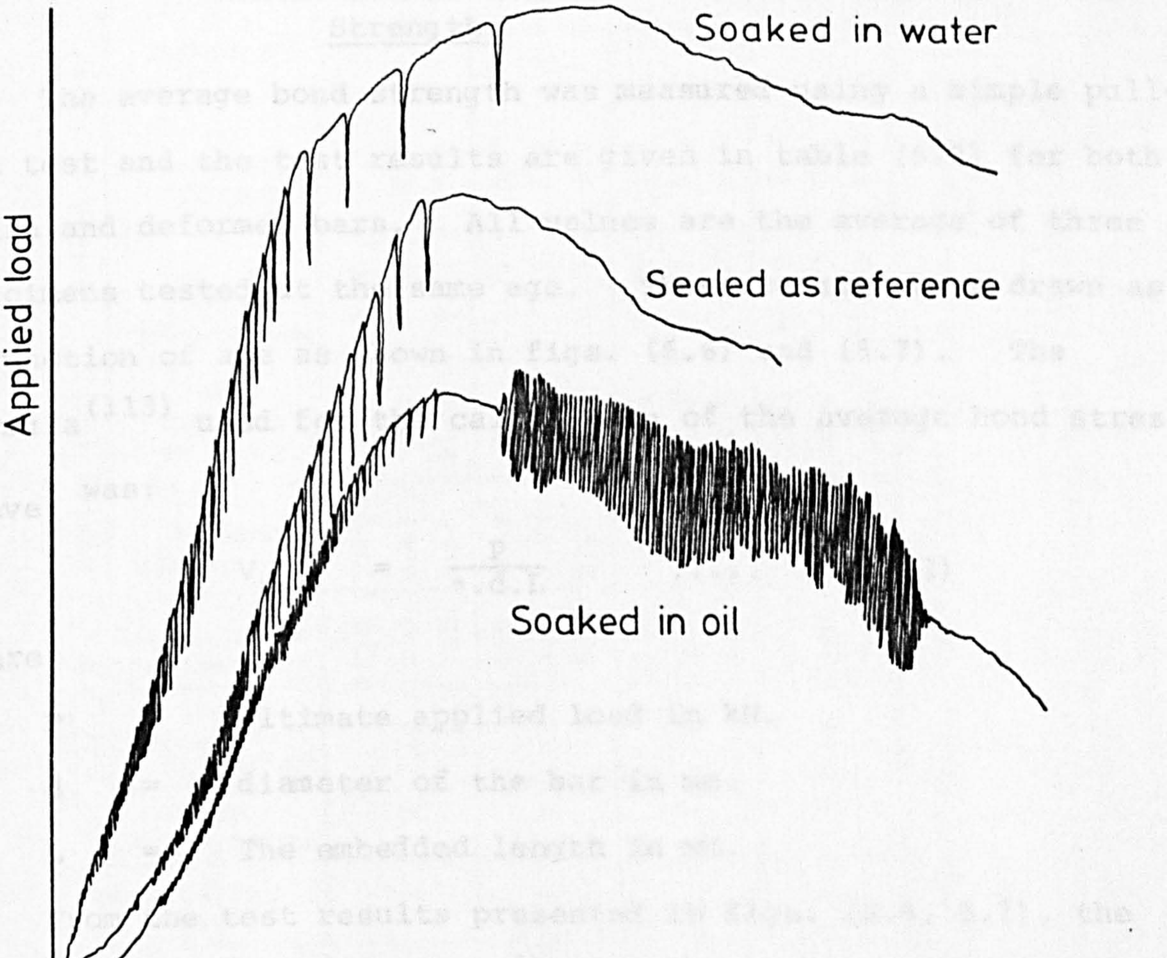


FIG.5-5 TYPICAL LOAD-SLIP CURVES AS DRAWN AUTOMATICALLY BY THE TESTING MACHINE - DEFORMED BARS

the bar being pulled out. This might be because of the effect of absorbed oil on the sliding friction between the bar surface and the surrounding oil saturated concrete. The initial slip is localized near the loaded end and some adhesion exists slightly beyond the slip zone, and most of the bar is entirely unstressed. As the pull is increased, the slip length increases and at failure the whole bar slips at a slow continuous rate. The presence of the absorbed crude oil seems to reduce the frictional resistance and cause a large slip to occur at a small load increment, thus the load-slip curve fluctuates continuously as the bar is being pulled out.

5.4.1.2 The Effect of Absorbed Oil on the Average Bond Strength.

The average bond strength was measured using a simple pull-out test and the test results are given in table (5.2) for both plain and deformed bars. All values are the average of three specimens tested at the same age. These results were drawn as a function of age as shown in figs. (5.6) and (5.7). The formula(113) used for the calculation of the average bond stress (V_{ave}) was:

$$V_{ave} = \frac{P}{\pi . d . L} \qquad \ldots\ldots \qquad (5.1)$$

where

P = Ultimate applied load in kN.

d = diameter of the bar in mm.

L = The embedded length in mm.

From the test results presented in figs. (5.6, 5.7), the following observations are made:

Type of Reinforcing bar	Mix Designation	Initial Curing		Storing period after drying (days)	Sealed (dry)		Soaked in water			Soaked in crude oil			
		Moist curing (days)	Drying in oven at 105°C (days)		Mean strength N/mm^2	Coeff. of Variation (V)	Mean strength N/mm^2	Coeff.of Variation (V)	Absorption % of Dry weight	Mean strength N/mm^2	Coeff.of Variation (V)	Absorption % of Dry weight	Degree of Saturation
Plain-bar	A	7	14	120	3.75	3.6	3.50	5.1	5.15	3.20	7.1	3.81	79.8
				300	3.63	9.0	3.75	6.2	5.48	2.84	8.1	3.89	81.5
				750	3.65	11.2	4.75	9.2	5.53	2.70	8.8	3.96	82.9
	B	7	14	120	3.58	6.1	3.71	5.5	5.29	3.11	9.6	3.98	81.0
				300	3.24	10.8	4.15	6.2	5.51	2.70	8.9	4.12	83.8
				750	3.35	12.1	5.05	3.3	5.58	2.45	7.9	4.29	87.3
Deformed-bar	A	7	14	120	15.64	3.1	16.20	3.6	5.32	13.47	2.80	3.86	81.3
				300	11.95	2.9	17.44	3.1	5.59	12.60	2.70	4.01	84.5
				750	15.10	2.6	19.02	2.8	5.68	12.30	2.40	4.22	88.9
	B	7	14	120	13.90	2.1	14.15	2.7	5.33	12.35	2.90	4.13	82.0
				300	13.58	2.5	14.85	3.2	5.62	11.60	1.40	4.23	84.0
				750	13.75	1.8	17.20	3.8	5.79	11.20	5.10	4.50	89.4

Table (5.2) Bond Strength of the oil soaked, sealed, and water saturated specimens for both plain and deformed bars.

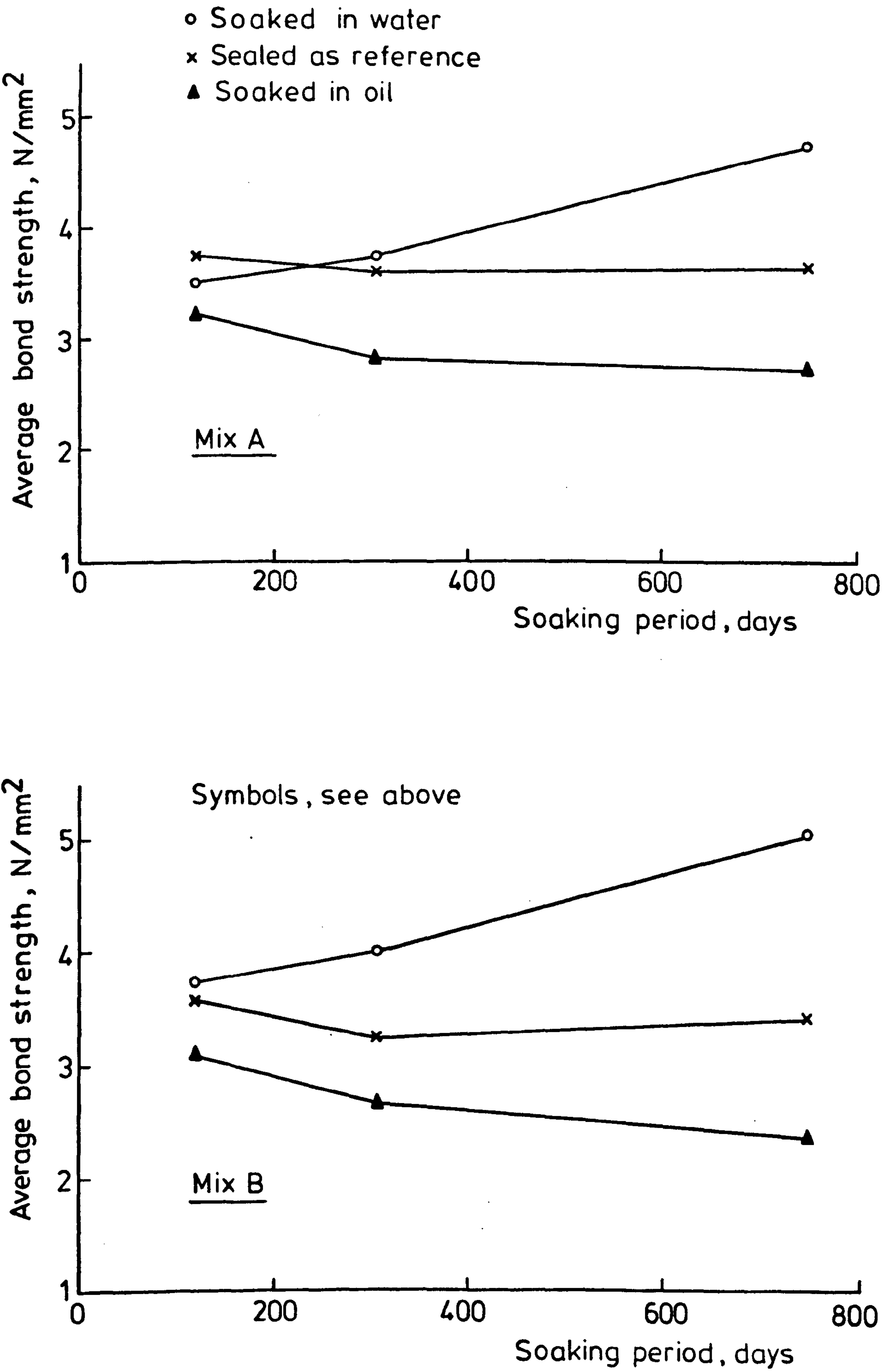


FIG. 5-6 AVERAGE BOND STRENGTH Vs. AGE, PLAIN BARS

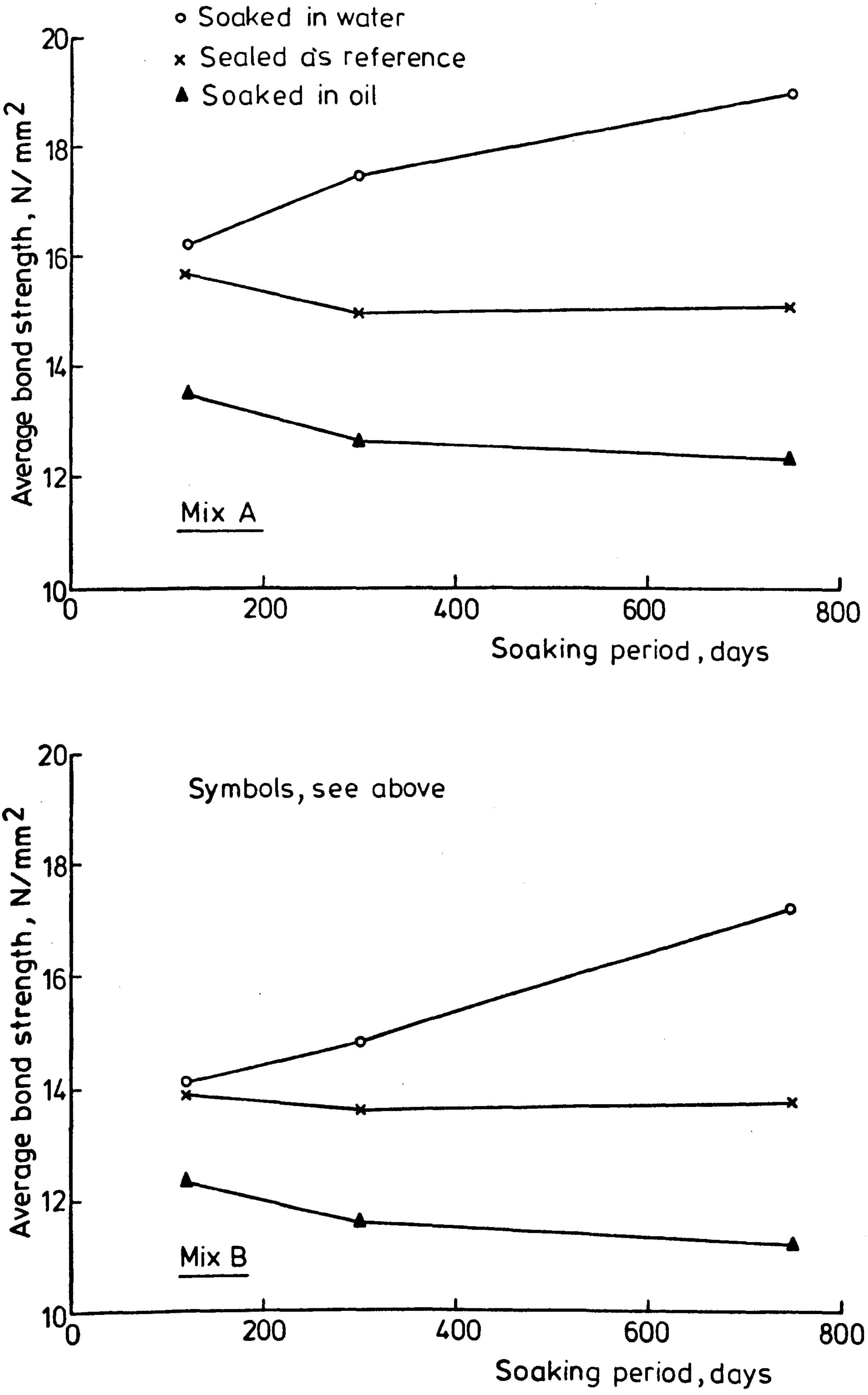


FIG. 5-7 AVERAGE BOND STRENGTH Vs. AGE, HIGH STRESS, HIGH BOND DEFORMED BARS

1. The average bond strength of the sealed specimens decreased with time. The maximum reduction was not significant at the 0.05 level and varied between 2.5-6.5 for plain bars and 1.5-3.5 for deformed bars within a storing period of 750 days.

2. The absorbed oil affected the bond strength for both plain and deformed bars in both concrete mixes. The reduction in the bond strength of a plain bar compared to that of the sealed control specimens tested at the same age,ranged between 14.5-26.5% for Mix A (with an oil content variation of 3.81-3.96% by weight), and was 13.5-26.5% for Mix B (with an oil content variation of 3.98-4.29% by weight). For deformed bars, the reduction ranged between 13.5-18.5% for Mix A (with an oil content variation of 3.86-4.22% by weight), and 13.5-19.5% for Mix B (with an oil content variation of 4.13-4.50% by weight). The larger reduction was shown by the specimens with higher porosity (Mix B, w/c = 0.6) because they absorbed more oil.

3. The bond strength of oil soaked specimens for Mixes A and B, and for both types of steel reinforcing bar, decreased with soaking period, i.e. increasing amounts of absorbed oil. This reduction was 15.5% for Mix A, and 21.0% for Mix B in the case of the mild round steel bar, and 8.5% for Mix A and 9.5% for Mix B in the case of the deformed bar. These values were calculated from specimens soaked in crude oil for a period of 750 days compared to that of the 120-day oil soaked strength.

4. The bond strength of the dry specimens resoaked in water showed a gradual increase with time. This increase varied between 35-40% for round mild steel bars and between 18-21% for the deformed steel bar within a soaking period of 750 days

compared to that of the 120-day water soaked strength.

5. The maximum increase in bond strength after soaking in water for 750 days compared to the companion sealed specimens, was 38-42% for the plain bar using Mix B and 25-26% for deformed bar using Mix A.

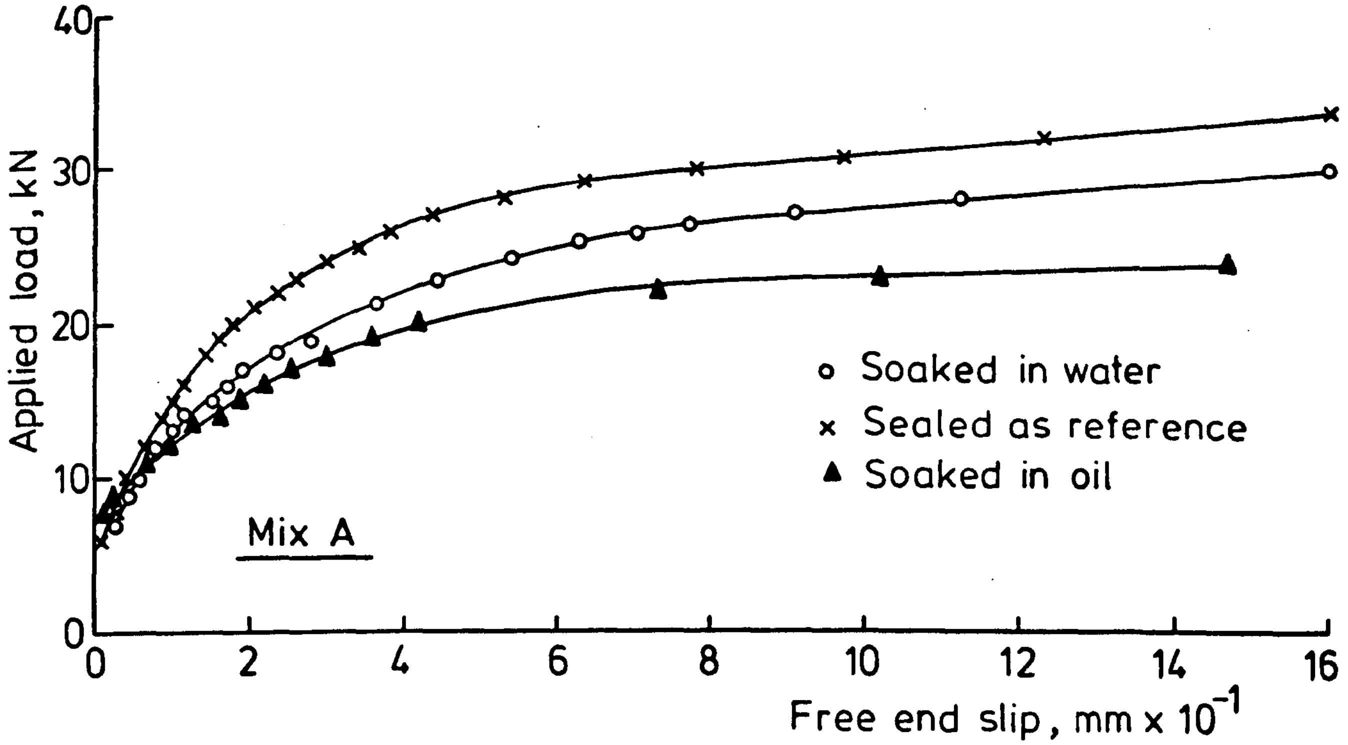
6. The bond strength of the deformed bar was about 4 times greater than the plain bar.

5.4.1.3 Load-Slip Relationship.

It was found that the slip gauge at the free end of the bar usually recorded only a very slight measurement until just before the instant of failure. After the maximum load was passed, the slip became so rapid that only a few readings were possible. These measurements were recorded and graphs of load-slip have been plotted as shown in figs. (5.8) to (5.13). These graphs enable further comparisons to be made between the behaviour of the different concretes.

Table (5.3) includes the value of bond stress corresponding to free end slip of 0.05, 0.10, 0.25, 0.50 mm along with the value of the total slip before failure. For deformed bars, the slip measurements at applied steel stress greater than 350 N/mm^2 were thought to be unrealistic because the bar was yielding. From tables (5.2, 5.3) and figs. (5.8) to (5.13), the following observations were made:

1. The graphs show that the general trend of the load-slip curves was the same for all the specimens. The rate of increase of the slip increased as the load was continued and near the ultimate load, the curve flattened until the load reached its maximum value and then the curve started to drop.



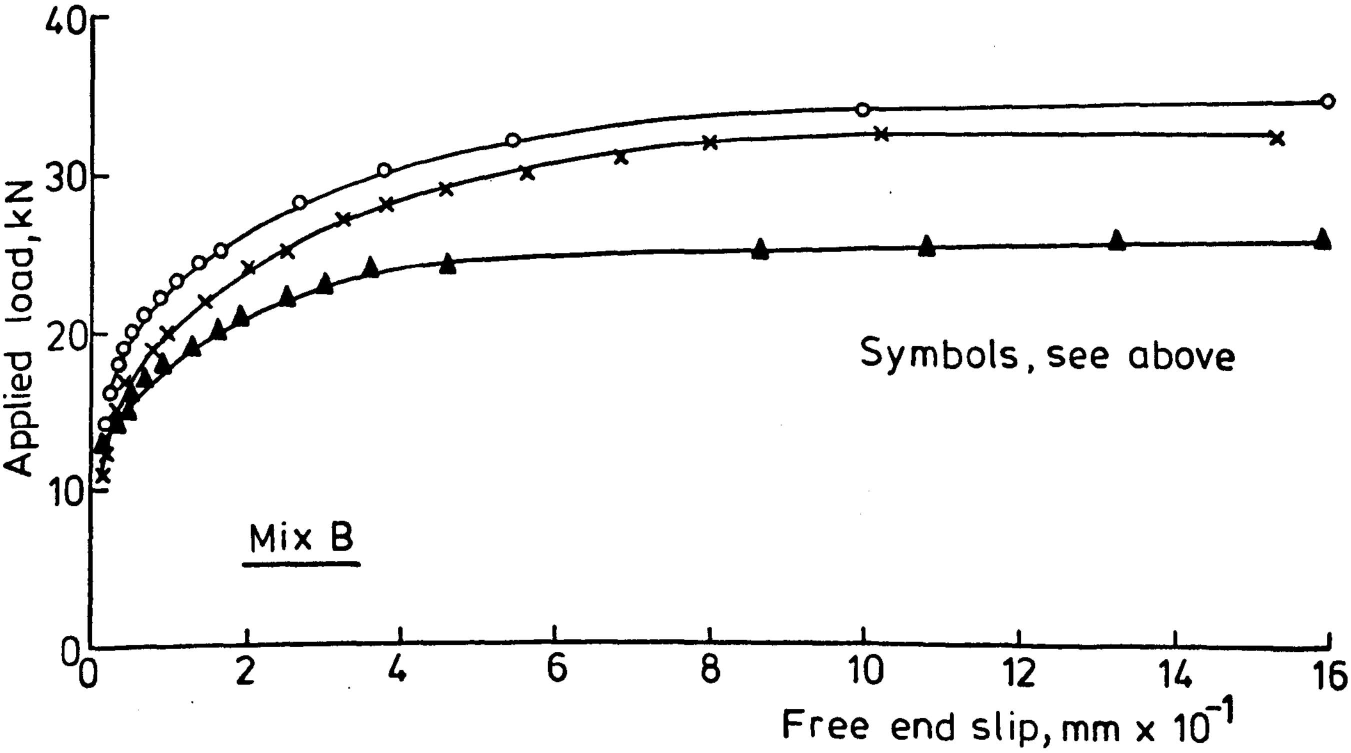


FIG. 5-8 APPLIED LOAD Vs. FREE END SLIP FOR PLAIN BAR SPECIMENS TESTED AT THE AGE OF 120 DAYS

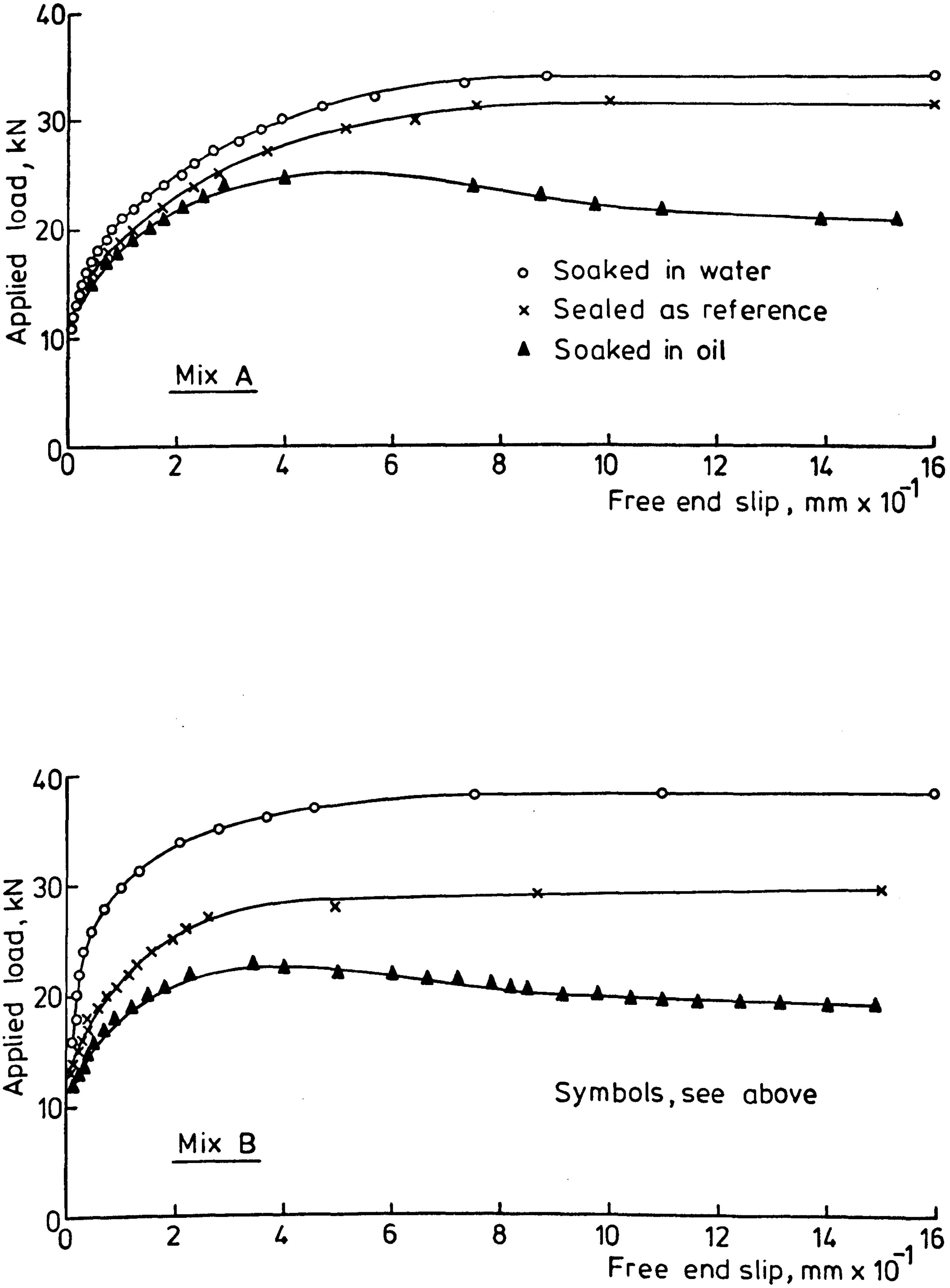


FIG. 5-9 APPLIED LOAD Vs. FREE END SLIP FOR THE PLAIN BAR SPECIMENS TESTED AT THE AGE OF 300 DAYS

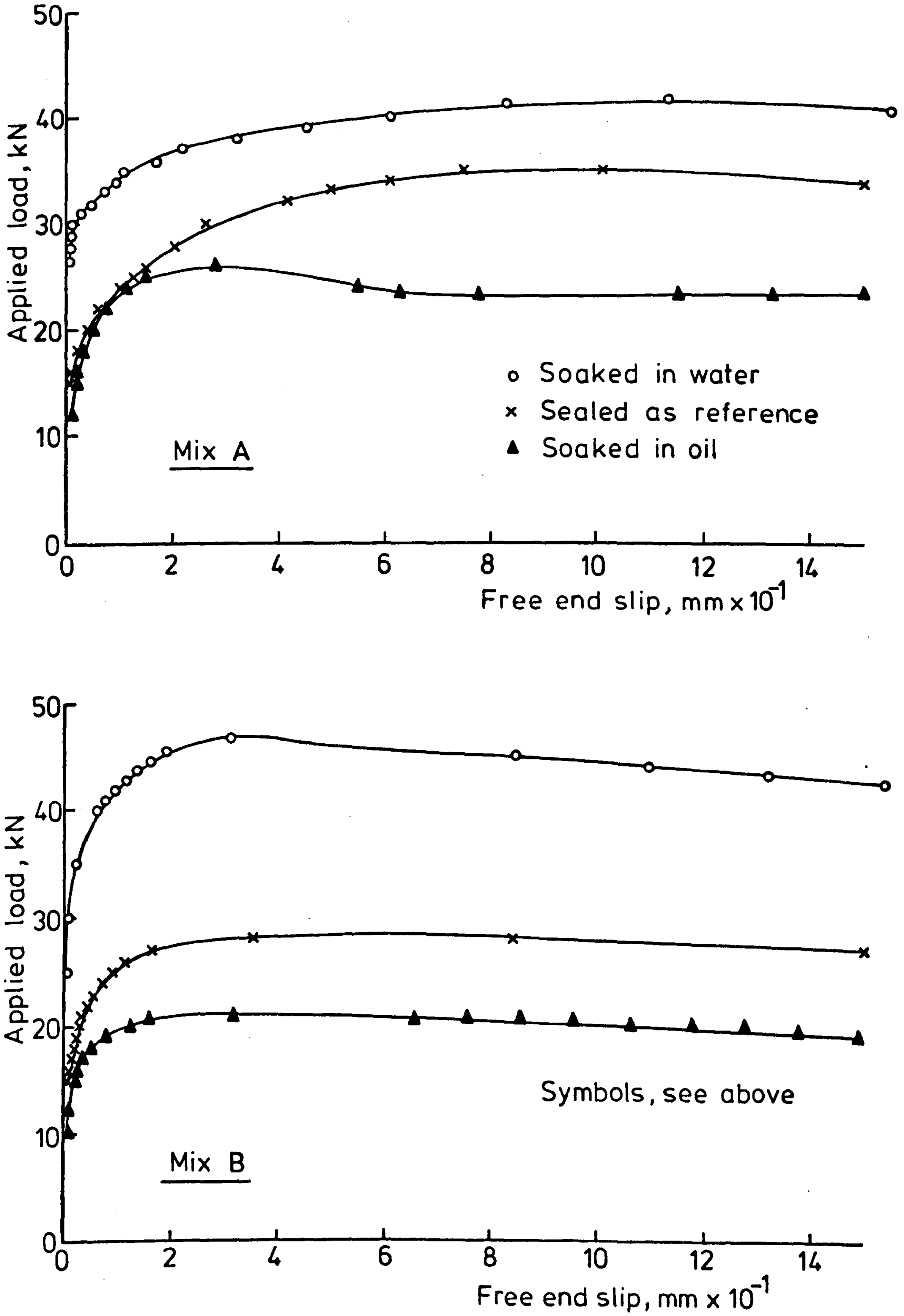


FIG. 5-10 APPLIED LOAD Vs. FREE END SLIP FOR THE PLAIN BAR SPECIMENS TESTED AT THE AGE OF 755 DAYS

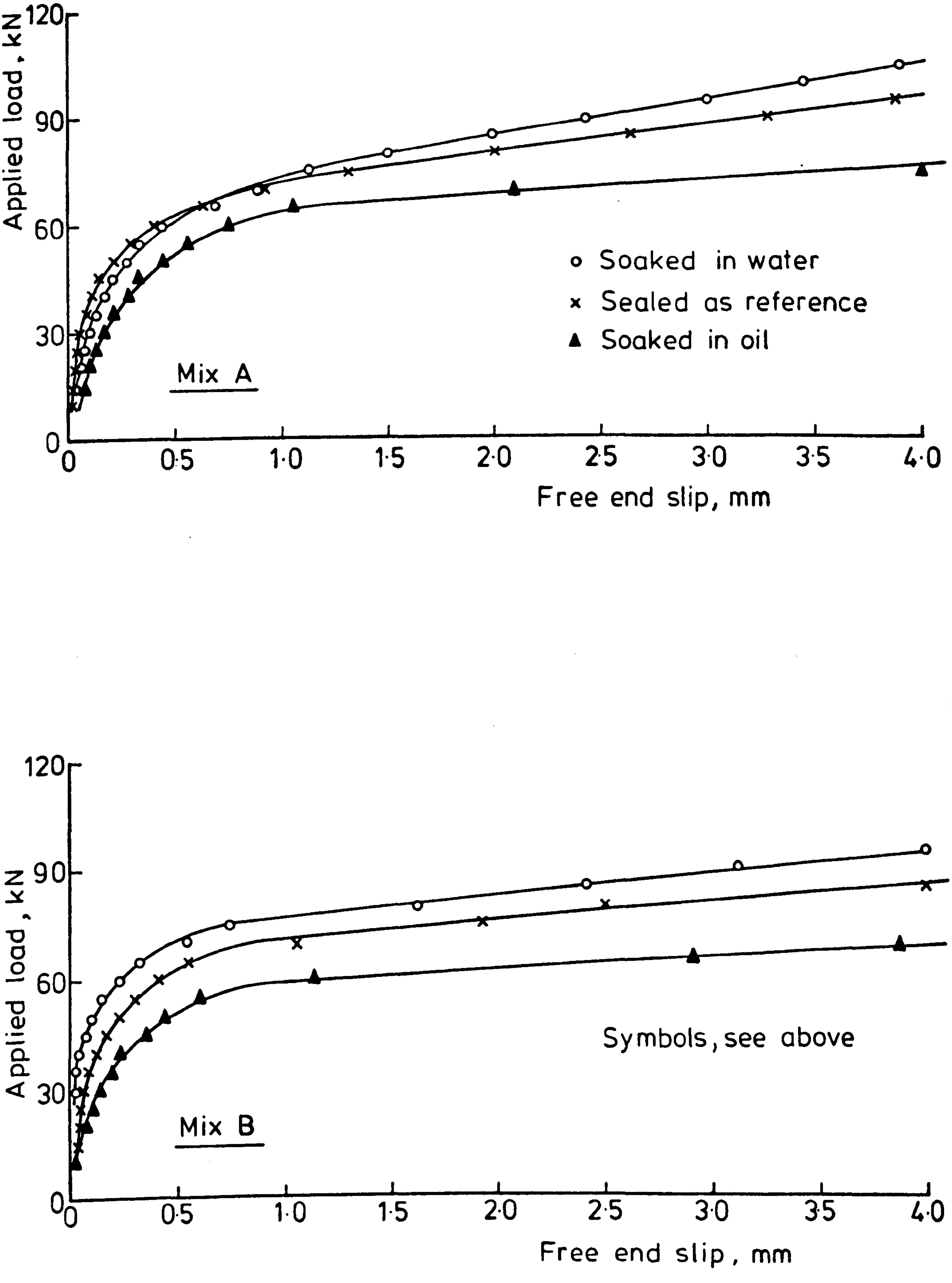


FIG. 5-11 APPLIED LOAD Vs. FREE END SLIP FOR THE DEFORMED BAR SPECIMENS TESTED AT THE AGE OF 120 DAYS

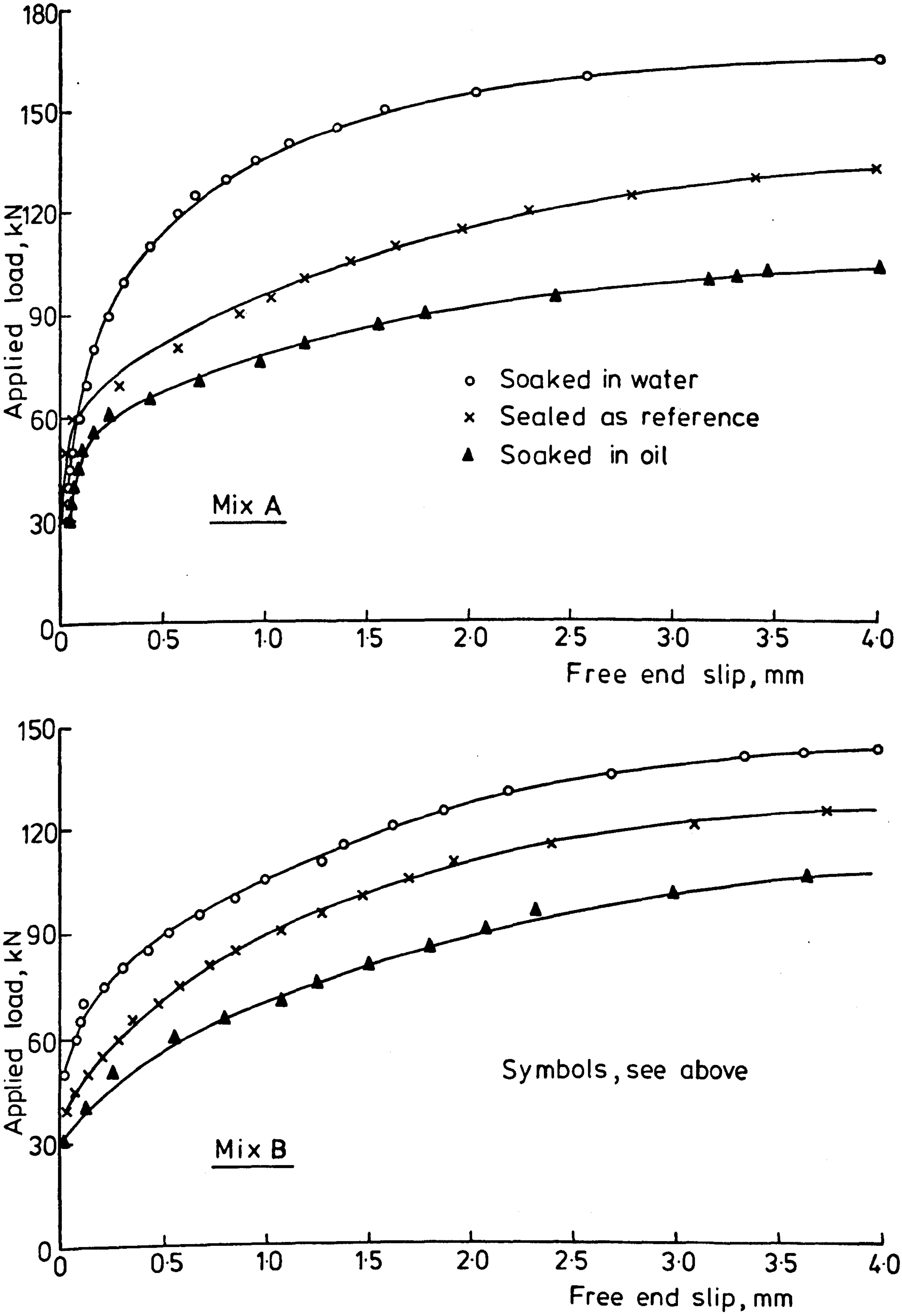


FIG. 5-12 APPLIED LOAD Vs. FREE END SLIP FOR THE DEFORMED BAR TESTED AT THE AGE OF 320 DAYS

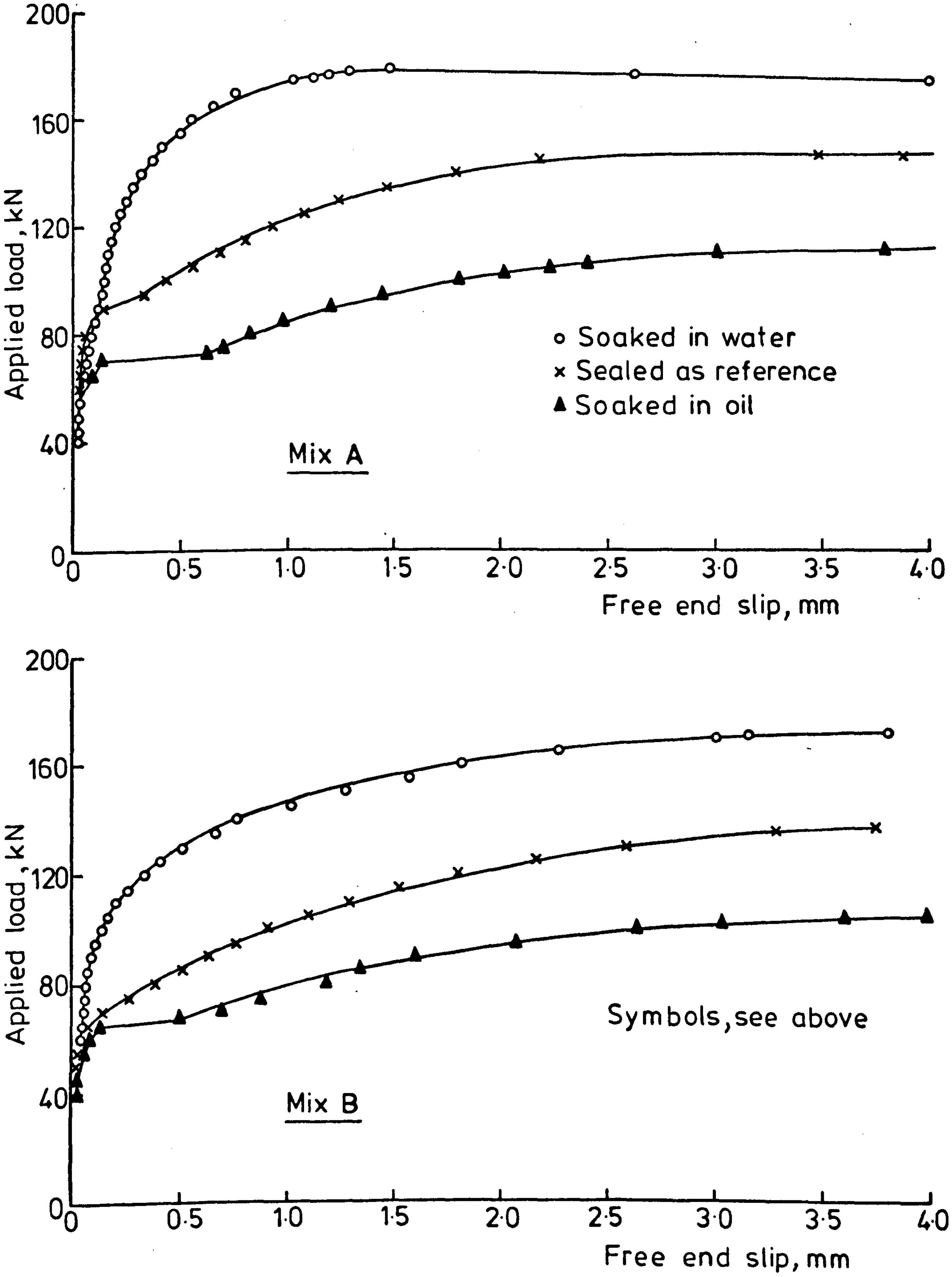


FIG. 5-13 APPLIED LOAD Vs. FREE END SLIP FOR THE DEFORMED BAR TESTED AT THE AGE OF 755 DAYS

Type of bar	Mix Designation	Ult. steel stress N/mm^2	Storing period (days)	Oil Saturated.					Sealed as reference.					Water saturated.				
				0.05 mm	0.10 mm	0.25 mm	0.50 mm	Total Slip mm	0.05 mm	0.10 mm	0.25 mm	0.50 mm	Total Slip mm	0.05 mm	0.10 mm	0.25 mm	0.50 mm	Total Slip mm
Plain-Bar	A	480	120	1.06	1.27	1.80	2.23	3.79	1.16	1.59	2.44	2.97	3.82	1.05	1.38	2.02	2.55	3.50
			300	1.59	1.96	2.44	2.65	0.60	1.75	2.02	2.60	3.02	2.60	1.86	2.17	2.81	3.34	2.10
			750	2.12	2.44	2.70	2.60	0.30	2.18	2.55	3.18	3.50	1.10	3.40	3.66	3.98	4.24	1.30
	B	480	120	1.59	1.91	2.30	2.60	3.79	1.85	2.12	2.65	3.18	3.22	2.10	2.38	2.92	3.34	3.16
			300	1.64	1.91	2.33	2.39	0.43	1.96	2.28	2.81	3.08	2.26	2.76	3.18	3.66	3.92	2.18
			750	1.91	2.12	2.28	2.28	0.30	2.39	2.71	2.97	2.97	0.35	4.03	4.51	4.93	4.96	0.35
Deformed-Bar	A	570	120	1.27	2.22	4.14	5.57	13.75	3.18	4.30	5.57	6.68	7.66	2.23	3.18	5.09	6.53	6.26
			300	3.66	4.78	6.36	7.16	5.65	5.88	6.52	7.64	8.59	4.70	4.77	7.00	9.55	12.10	4.70
			450	6.36	7.22	7.43	7.64	5.48	8.49	9.34	9.76	11.63	4.42	7.00	9.12	13.79	16.55	1.60
	B	570	120	1.27	2.71	4.30	5.41	15.75	3.20	4.14	5.55	6.65	9.24	4.70	5.41	6.53	7.48	6.38
			300	3.66	3.98	5.25	6.05	4.99	4.45	4.93	6.21	7.48	4.34	6.21	7.00	8.28	9.55	4.39
			750	5.73	6.79	7.00	7.22	7.28	6.79	7.22	7.85	9.12	4.25	7.43	9.55	12.10	13.79	4.71

Plate (5.3) Average bond strength at different free end slip and the total free end slip at failure for both plain and deformed bars.

2. At low loads, the 120-day oil saturated specimens showed low slip values compared to the specimens with other storage conditions. This might be due to the shrinkage produced when the dry specimens were soaked in crude oil as described in Chapter 4. These stresses acted centrally towards the bar, pressing the cement paste into the surface of the bar. But as the spplied load increased, this effect was reduced and the resistance to withdrawal would be supplied mainly by the frictional drag and mechanical wedging which depended upon the character and type of surface texture as discussed later in section (5.4.1.4).

3. From figs. (5.8) to 6.13) and table (5.3), there were two differences between water and oil stored concrete specimens:

(a) for the same value of slip, the water stored specimens developed higher bond stresses than oil stored specimens,

(b) for both the sealed and water stored specimens, the plain bars had more free end slip before failure compared to those specimens saturated with crude oil. For the deformed bar this effect was reversed.

4. For the dry specimens soaked in water, the average bond strength was increased as the soaking period increased and the specimens had less free end slip before failure as the average bond strength increased. This behaviour showed that bar slip was greater in weak concrete.

5. The total amount of free slip before failure for oil soaked specimens reduced as the soaking period increased. This might be due to the shrinkage of the dry specimen as it was soaked in oil, section (4.4.9). This shrinkage would increase the adhesive resistance between reinforcing bar and surrounding concrete.

6. The critical bond stress corresponding to a free end slip of (0.05 mm) was the lowest for the oil saturated specimens. The critical bond stress for all the specimens and for both concrete mixes, increased as the storage period increased.

7. Slip of the deformed bars was sometimes due to the crushing of the concrete under the bar ribs. The crushed zones are seen in plate (5.5) showing the sawn surfaces along the diameter of a specimen.

5.4.1.4 Mechanism of Bond failure in Pull-Out Test.

The reference samples, which were sealed in polythene bags and stored as control specimens, showed a reduction in bond strength with time probably because of slow crack propagation during the cooling process(63) after the specimen had been oven dried. Separation of the bar and the surrounding concrete will occur in the vicinity of the crack, and in the case of plain bars the separation would mean complete loss of bond in that region. In the case of a deformed bar, the reduction in the compressive strength of the mortar and concrete during storage of the dry specimens, section (4.4.1), would also cause a considerable reduction in the bond strength.

The bond strength of water soaked specimens was higher than that of the sealed specimens and there was a continuous increase in the bond strength as the soaking period increased. This might be due to the fact that hydration resumed again after being stopped during the drying process(74). During hydration, nearly formed gel and crystalline masses grow into the steel filling all the irregularities of the steel surface and thereby increasing both the adhesive area and the frictional resistance. This is in line with Plowman's(110) proposition, which states that the breakdown of bond is by shear of a cylindrical surface in the mortar which infiltrates the micro-indentations of the bar surface and once slip occurs the adhesive/micro-keying fails and only friction between the bar surface and concrete remains. For the deformed bar, the increase in the compressive strength of dry specimens upon soaking in water for long periods causes an increase in the bearing pressure required to crush the concrete between the ribs and thus increases the bond strength.

The bond strength of the oil soaked specimens was lower than that of the sealed and water saturated specimens as shown in figs. (5.6) and (5.7). The variation in the reduction depends on the type of reinforcing bar and the amount of absorbed oil. Table (5.4) shows the reduction in the bond strength of oil soaked specimens compared with the sealed specimens. These results show that the reduction increases with the amount of absorbed oil.

Type of reinforcing bar	Storing period (days)	Mix A			Mix B		
		Sealed (dry)	Soaked in water	Soaked in oil	Sealed (dry)	Soaked in water	Soaked in oil
plain bar	120	100.0	93.3	84.0	100.0	103.6	86.9
	300	96.8	100.0	75.7	90.5	115.9	79.3
	750	97.3	126.6	73.6	96.4	141.1	77.1
deformed bar	120	100.0	103.6	86.1	100.0	101.8	88.9
	300	95.6	111.5	80.6	97.7	106.8	83.5
	750	96.5	121.6	78.6	98.9	123.7	80.6

Table (5.4) The percentage of change in the bond strength as a percentage of sealed specimens tested at the age of 120 days.

In the case of plain bar, the oil will cause a significant reduction in the frictional resistance between the steel bar and the concrete especially near the ends of the specimen, thus lowering the bond strength. For deformed bars, bond(109,113) depends primarily on mechanical interlocking and the ribs of the bar restrain slip by bearing against the concrete between the ribs producing normal (direct) stresses under the bearing surfaces of the ribs, and shear stresses at the outer perimeters of the concrete bracket between two ribs(122). The sliding resistance developed by the bar between the ribs constitutes only a small proportion of the total resistance capacity. The magnitude of this proportion depends essentially upon the roughness of the surface of the bar, and upon the cylindrical bar surface area in relation to the rib surface area(122). The presence of oil is believed to increase the relative movement between the ribs and the surrounding concrete because it reduces the frictional resistance. The reduction in the compressive strength of oil saturated mortar and concrete, section (4.4.1), causes a reduction in both the bearing and shearing stresses required to crush the concrete between the ribs and eventually reduces the bond strength below that of sealed or water saturated specimens.

5.4.2 Modified Pull-Out Test.

5.4.2.1 General Behaviour.

The specimens were loaded in increments of load up to failure during which the strain gauges and the dial gauge readings were recorded. At the early stages of loading, the first few strain gauges near the loaded end of the bar registered load and as the applied load was increased, gauges successively further along the bar began to register. Typical strain gauge results are plotted against the applied stress as shown in figs. (5.14, 5.15) for all storage conditions.

Bond failure at ultimate load was indicated by the sudden drop of the load in the bar except for water saturated specimens which fractured at the mid section when the bond strength reached the maximum tensile strength of concrete at that section. For sealed and oil saturated specimens, the load decreased slowly as the bar was pulled from the surrounding concrete. The load then became constant at about three fourths of the maximum load and the bond resistance started to increase again but did not reach its maximum value. The specimen failed by breaking into two halves at the mid section before the bar was completely pulled out as shown in plate (5.6). This failure was classified as bond failure because of the large slip at the loaded end of the bar.

The maximum load producing failure of the pull-out specimen was recorded as an average of three specimens tested at the same age. Test results are shown in table (5.5) for both concrete mixes. The effect of absorbed oil was significant in both mixes. The maximum reduction of the calculated average bond strength of oil saturated concrete specimens, compared to that of sealed specimens, was between 7-11.5% (with an oil content variation of 3.92-4.24% by weight). These values are calculated from specimens soaked in crude oil for a period of 8 months, sufficient to reach full saturation as predicted in the previous chapter - section (4.4.12).

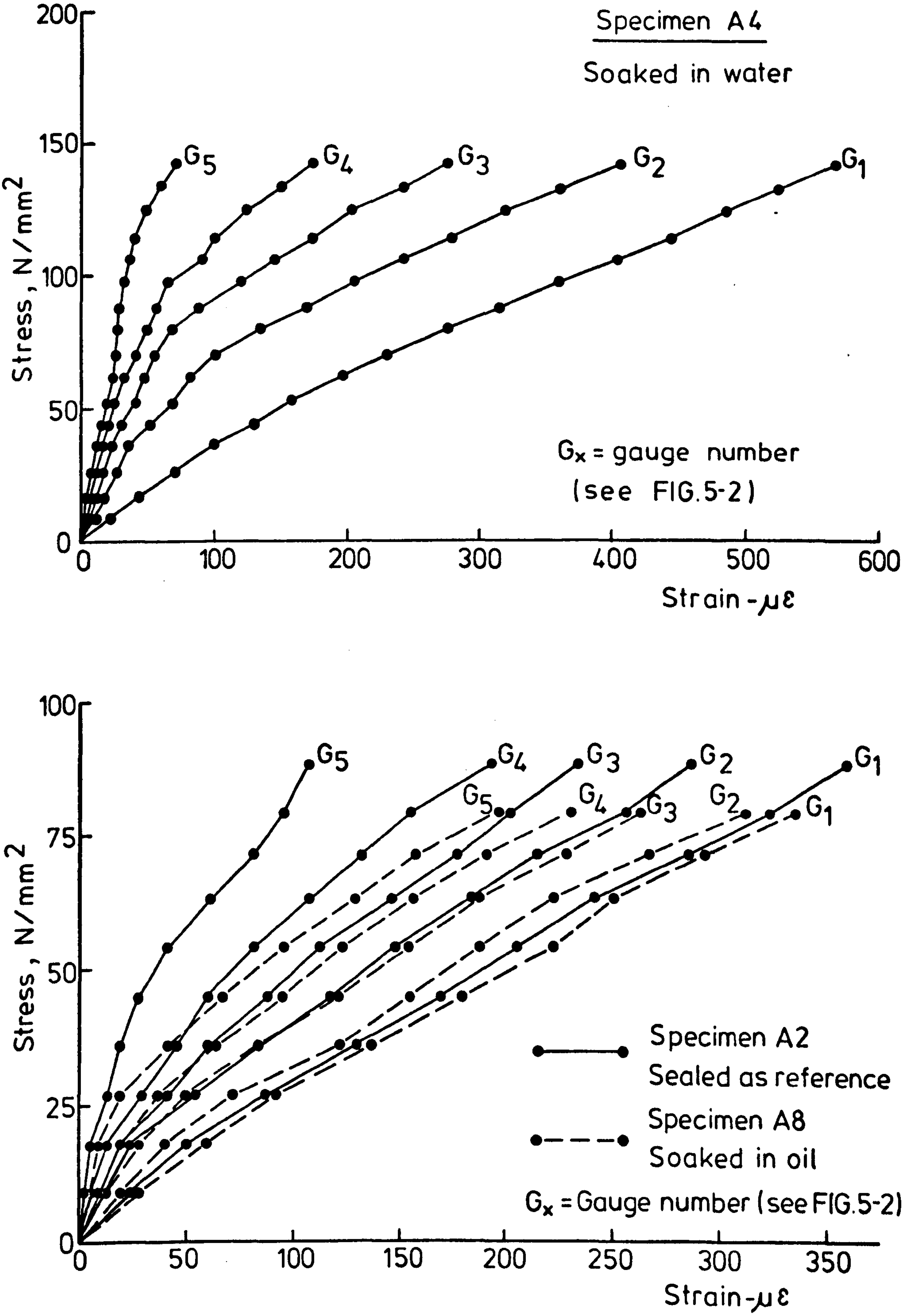


FIG. 5-14 APPLIED LOAD Vs. STRAIN GAUGE READINGS FOR THE SPECIMENS OF THE THREE DIFFERENT STORAGE CONDITIONS, MIX A

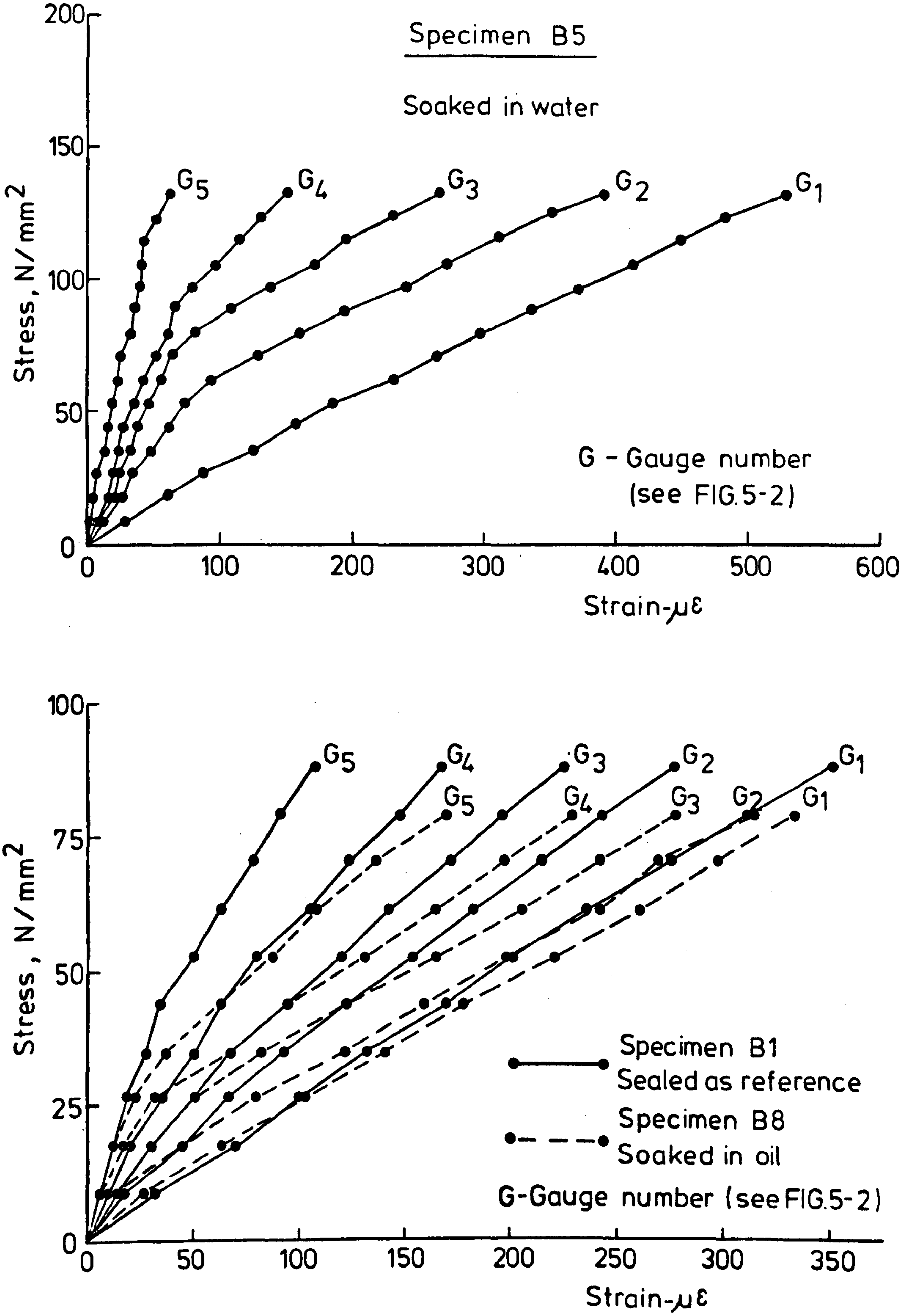


FIG.5-15 APPLIED LOAD Vs. STRAIN GAUGE READINGS FOR THE SPECIMENS OF THE THREE DIFFERENT STORAGE CONDITIONS, MIX B

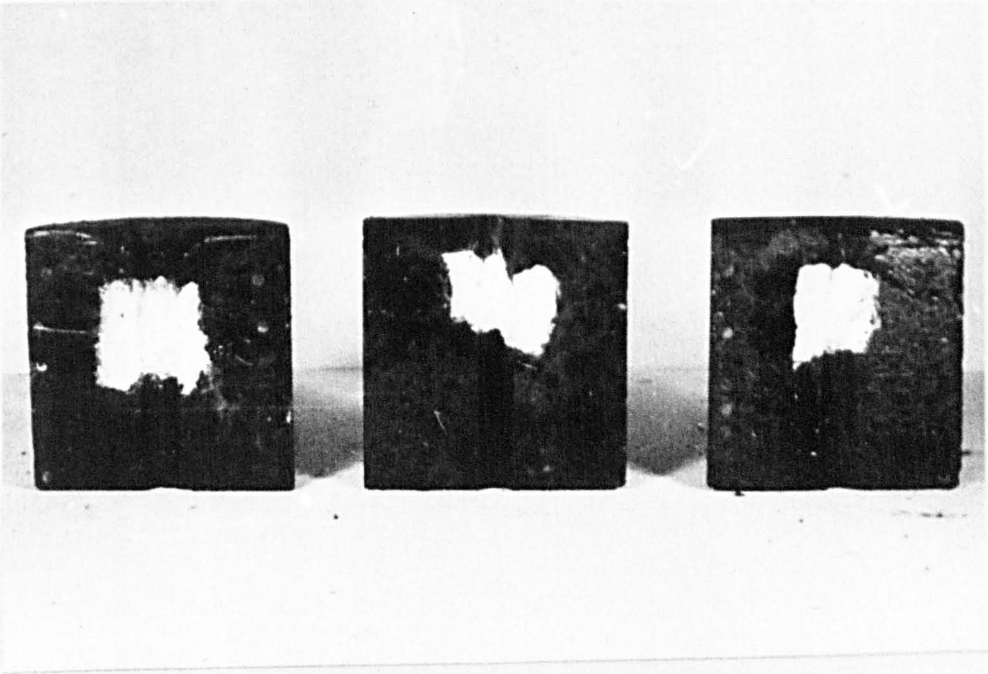


PLATE. 5-5. PENETRATION OF OIL THROUGH THE PULL-OUT SPECIMENS AS SHOWN UNDER U.V.LAMP

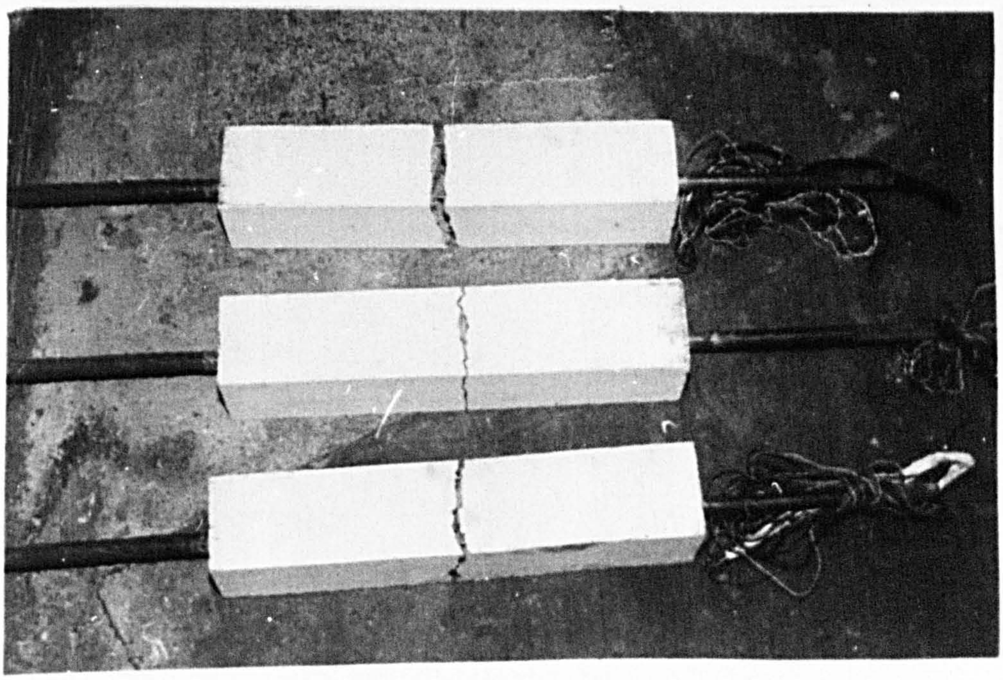


PLATE. 5-6. MODIFIED PULL-OUT TEST, TYPE OF FAILURE

Mix Designation	Initial Curing		Storing Condition after drying	Storing period (days)	Absorption % of Dry Weight	Degree of Saturation %	Bond Stress (N/mm^2)		Total slip at failure
	Moist Curing (days)	Drying at 105°C (days)					at loaded end slip of 0.025 mm	ult. average bond stress	
A	7	14	Sealed (dry)	240	0.48	10.0	0.46	1.67	0.29
			Soaked in oil	240	3.92	84.5	0.30	1.45	0.38
			Soaked in water	240	4.45	95.4	1.11	2.37	0.20
B	7	14	Sealed (dry)	240	0.48	9.2	0.63	1.59	0.18
			Soaked in oil	240	4.24	82.7	0.37	1.39	0.25
			Soaked in water	240	5.80	99.7	1.30	2.23	0.14

Table (5.5) Percentage of oil absorption, the average ultimate bond strength, and the critical bond stress at free end sip of 0.025 mm for the modified pull-out test.

The average bond strength of the dry specimens soaked in water for the same period as the oil saturation specimens, was about 32-40% higher than that of the sealed control specimens.

5.4.2.2 Load - Slip Relationship.

Typical load - slip curves measured by the dial gauges for some of the specimens of group A and B are shown in fig. (5.16). For practical reasons the dial gauge was attached 51 mm from the face of the concrete,it was then necessary to subtract from the dial gauge reading the elastic elongation of 51 mm of the bar to obtain the slip. This elongation is determined by the stress in the bar and the known modulus of elasticity of the steel bar. The slip values shown in fig. (5.16) are the net values after correction, which suggest the following observations:

1. For the sealed and the oil saturated specimens, the slip started immediately after applying the load. While for the water cured specimens, there was no slip at the loaded end until the stress was approximately 0.56-0.70 N/mm^2.
2. For the same slip, the oil saturated specimens developed lower bond strength than that of the sealed specimens. The dry specimens soaked in water developed the highest bond strength.
3. Both the sealed and oil saturated specimens have more loaded end slip before failure compared to that of water saturated specimens tested at the same age.
4. The critical bond stress corresponding to a loaded end slip of 0.001 in. (0.025 mm) was higher for water saturated

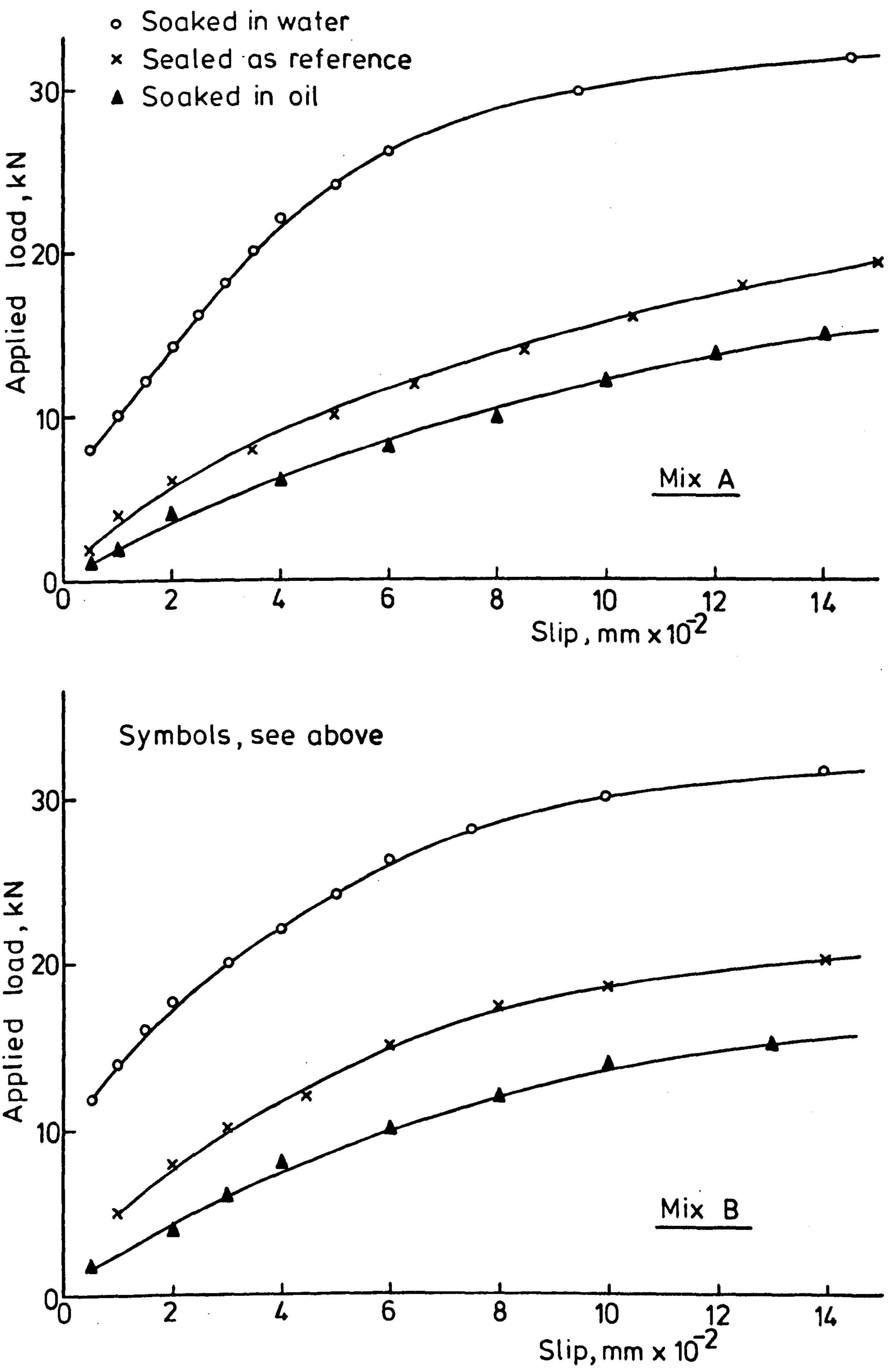


FIG. 5-16 APPLIED LOAD Vs. LOADED END SLIP FOR THE MODIFIED PULL-OUT TEST

specimens than for the sealed and oil saturated specimens. The value of the critical bond stress for each type of storage condition is given in table (5.5) along with the value of the ultimate average bond stress.

5.4.2.3 The Effect of Storage Conditions on the Load-Transfer and Bond Stress Distribution.

Force transfer by bond between a steel bar and concrete was measured on concentric tension pull-out specimens stored at different conditions as described in section (5.3.2). The stresses along the bar were obtained by direct calculations from the measured strains, through the use of the average gauge factor and the modulus of elasticity of the steel bar since the steel stresses were within the elastic range.

Typical curves for the force variation in the bars with increasing applied loads are shown in figs. (5.17) to (5.19). From these figures, it can be noticed that all the specimens had almost similar load transfer curves; the force in the bar decreased from the loaded end towards the middle part of the specimen. For relatively small loads, a major part of the load acting on the bar was transferred to the surrounding concrete through a short segment near the loaded end of the bar. The remaining part of the bar was left with a very low stress. As the load on the pull-out specimen increased, the slope of the load transfer curves near the loaded end decreased and the curves approached a straight line, while for the inner region the slope of the curve increased due to the additional stresses picked up as the applied load increased. The movement of the load distribution toward the embedded end continued

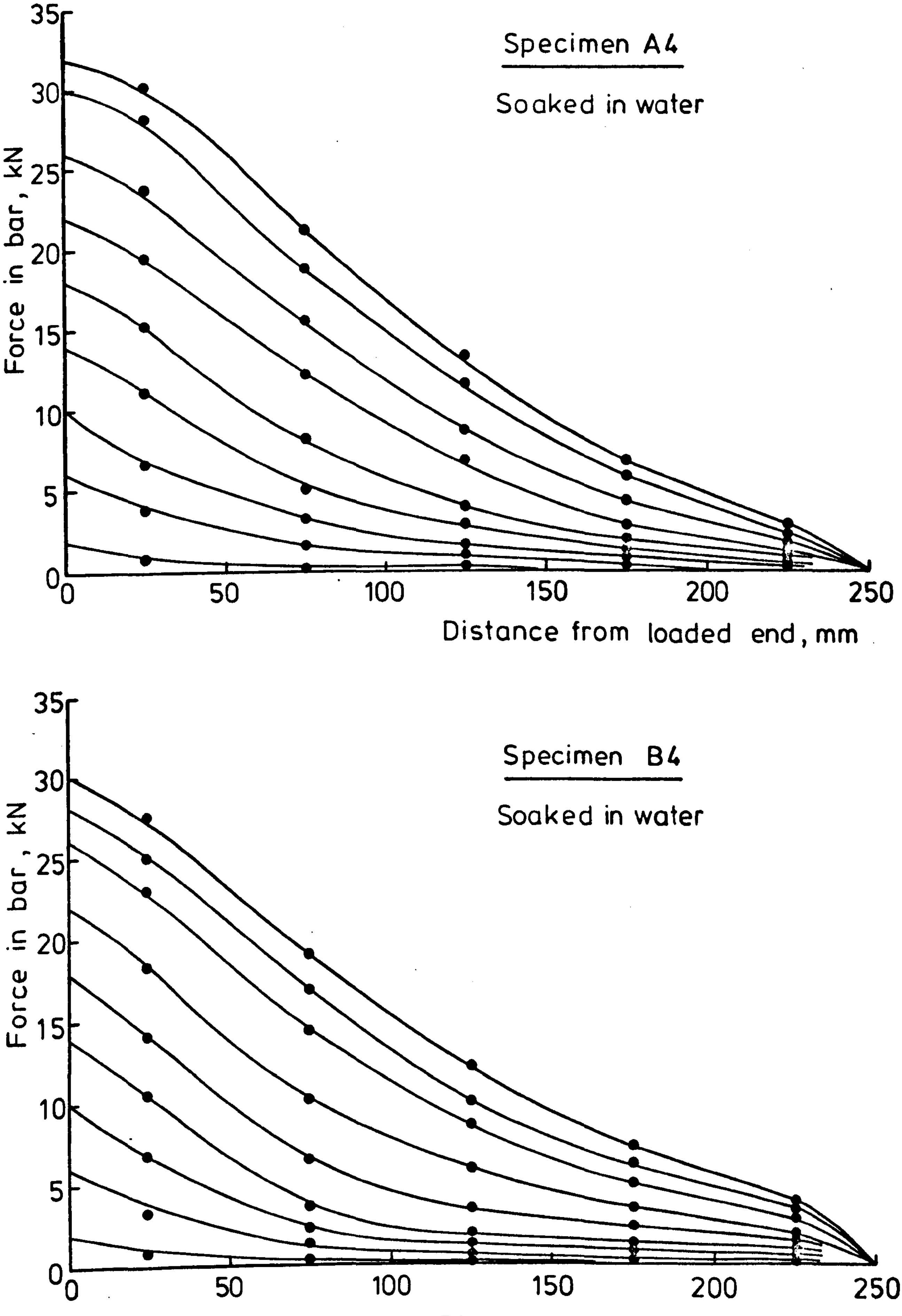


FIG. 5-17 LOAD DISTRIBUTION ALONG THE BAR FOR THE WATER SATURATED SPECIMENS

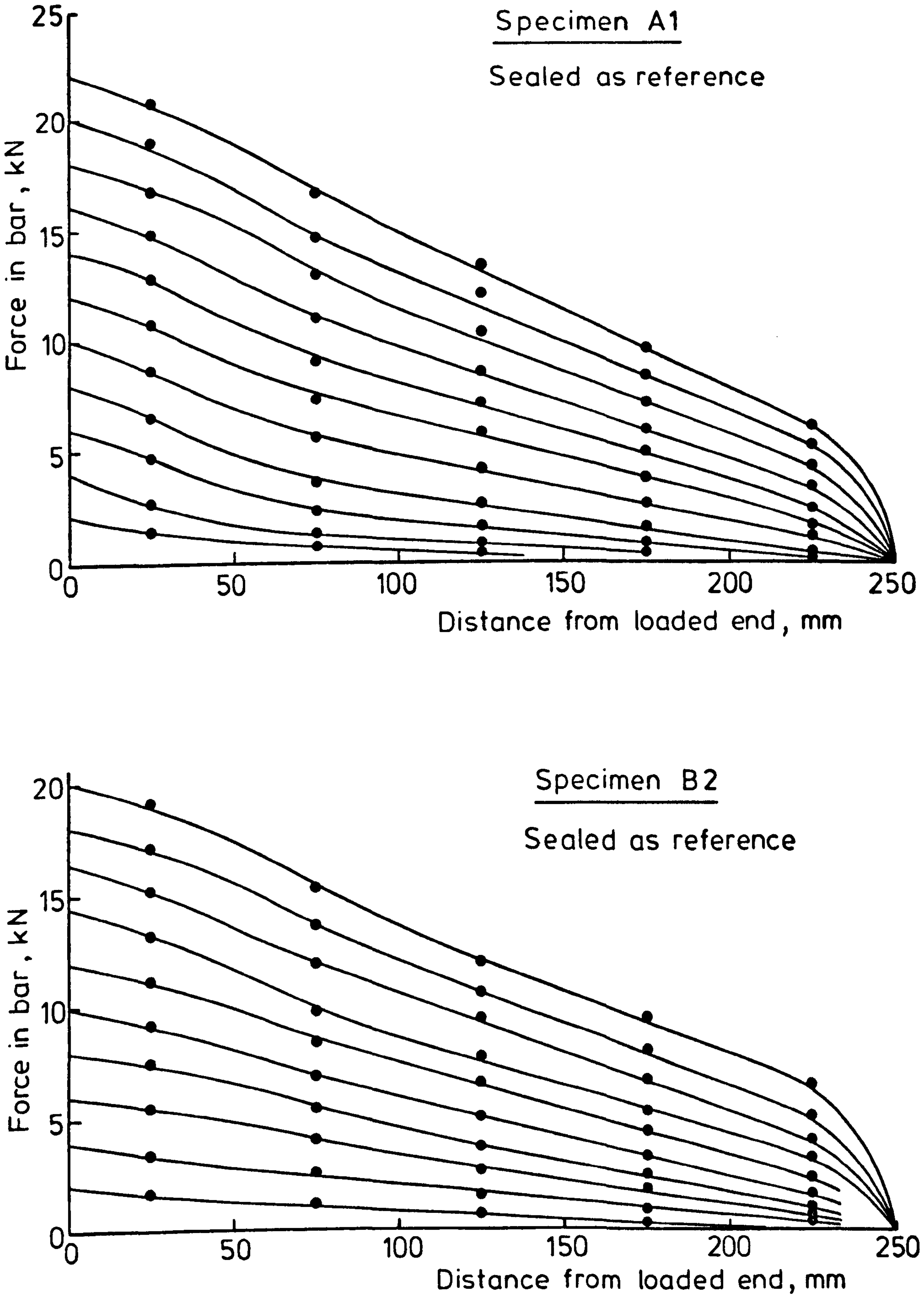


FIG. 5-18 LOAD DISTRIBUTION ALONG THE BAR FOR THE SEALED SPECIMENS

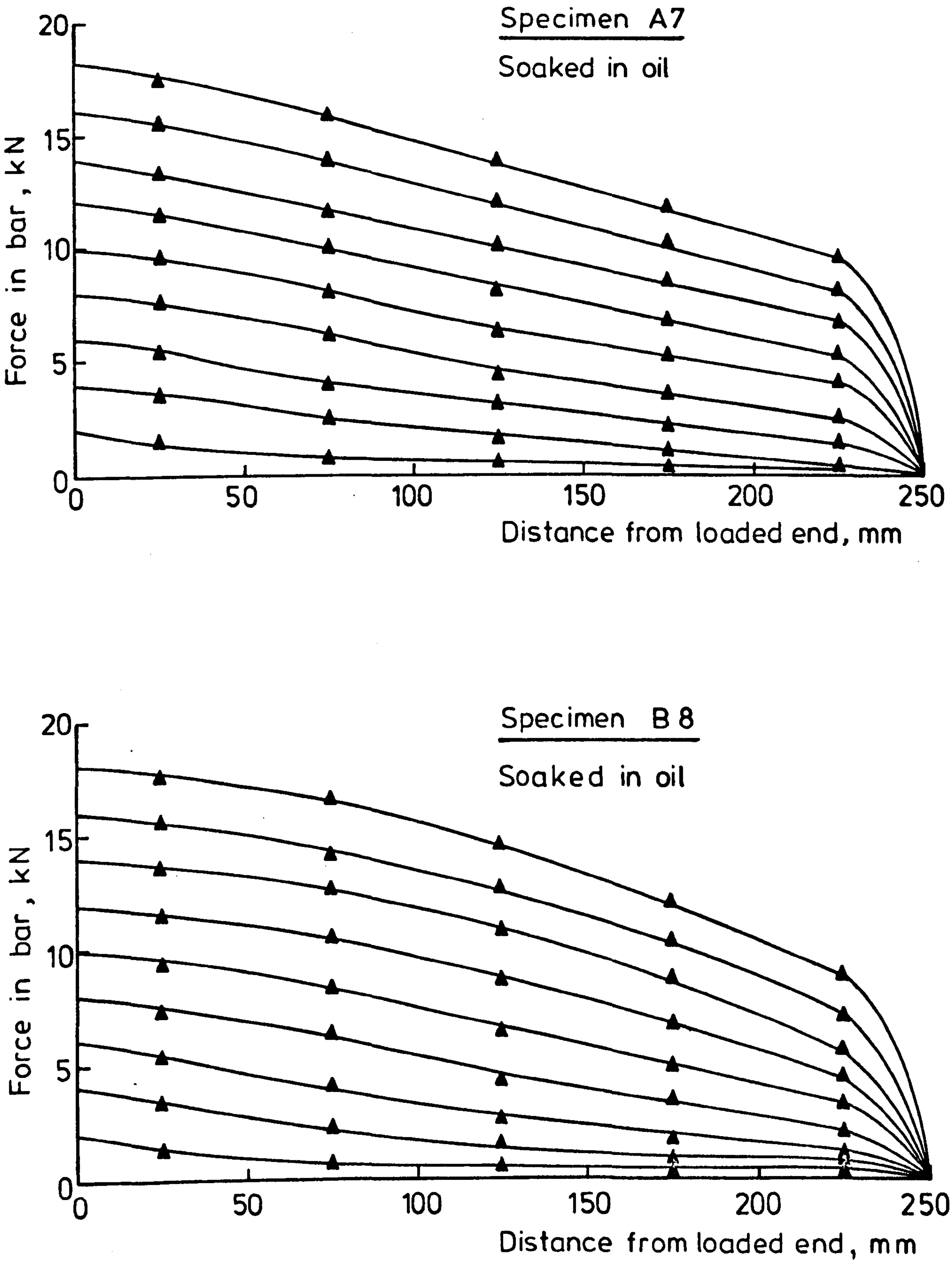
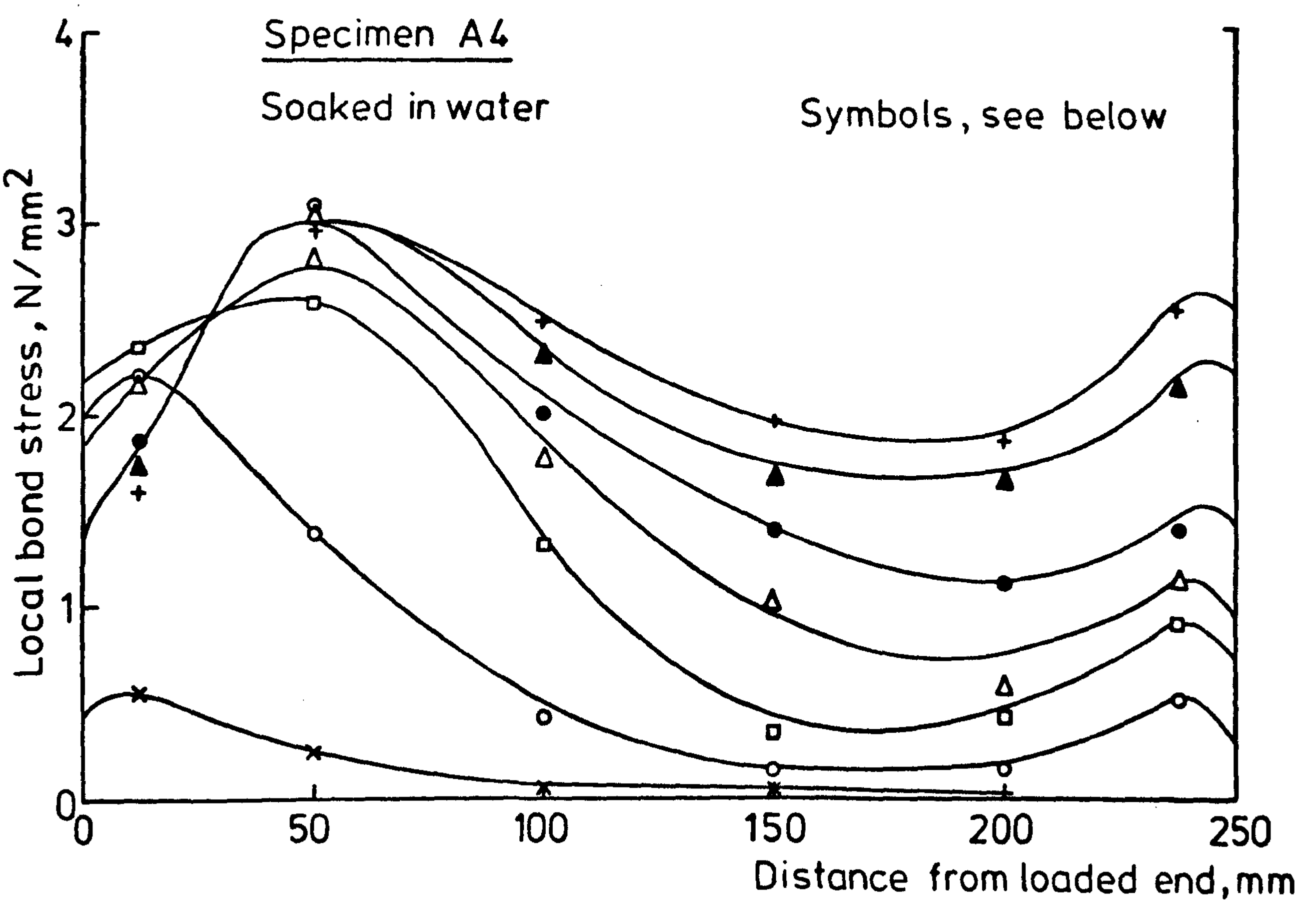


FIG. 5-19 LOAD DISTRIBUTION ALONG THE BAR FOR THE OIL SATURATED SPECIMENS

with the increasing value of applied load until the bond capacity of the bar was reached and total failure resulted.

The bond stress distribution curves, shown in figs. (5.20) to (5.25), have been obtained from the load transfer curves through the determination of the force difference between two successive gauge points. This difference, when divided by the surface area of the bar, represents the average local bond stress for the interval between these gauge points. This value was then plotted at the centre of the interval. The unit bond stress values at the gauge points were also determined from the slope of the load transfer curves and were plotted at the appropriate locations. The curve was then drawn through these points to give a good picture of the bond stress distribution along the bar.

From the bond stress distribution shown in figs. (5.20) to (5.25), it can be seen that for small loads, the maximum bond stress occurred near the loaded end of the bar reducing to near zero at some distance from that face. The increase in bond stresses near the loaded end continued until reaching a maximum limiting value after which they started to decrease approaching a constant value. The decrease in the bond stresses near the loaded end was accompanied by an increase in the bond stresses along the remaining part of the bar. Thus it is apparent from the bond stress distribution curves that the bond stress at any point along the bar varied with the load acting on the specimen. This variation depended on the distance of the point from the loaded end and on the storage condition of the tested specimen. This agreed with the



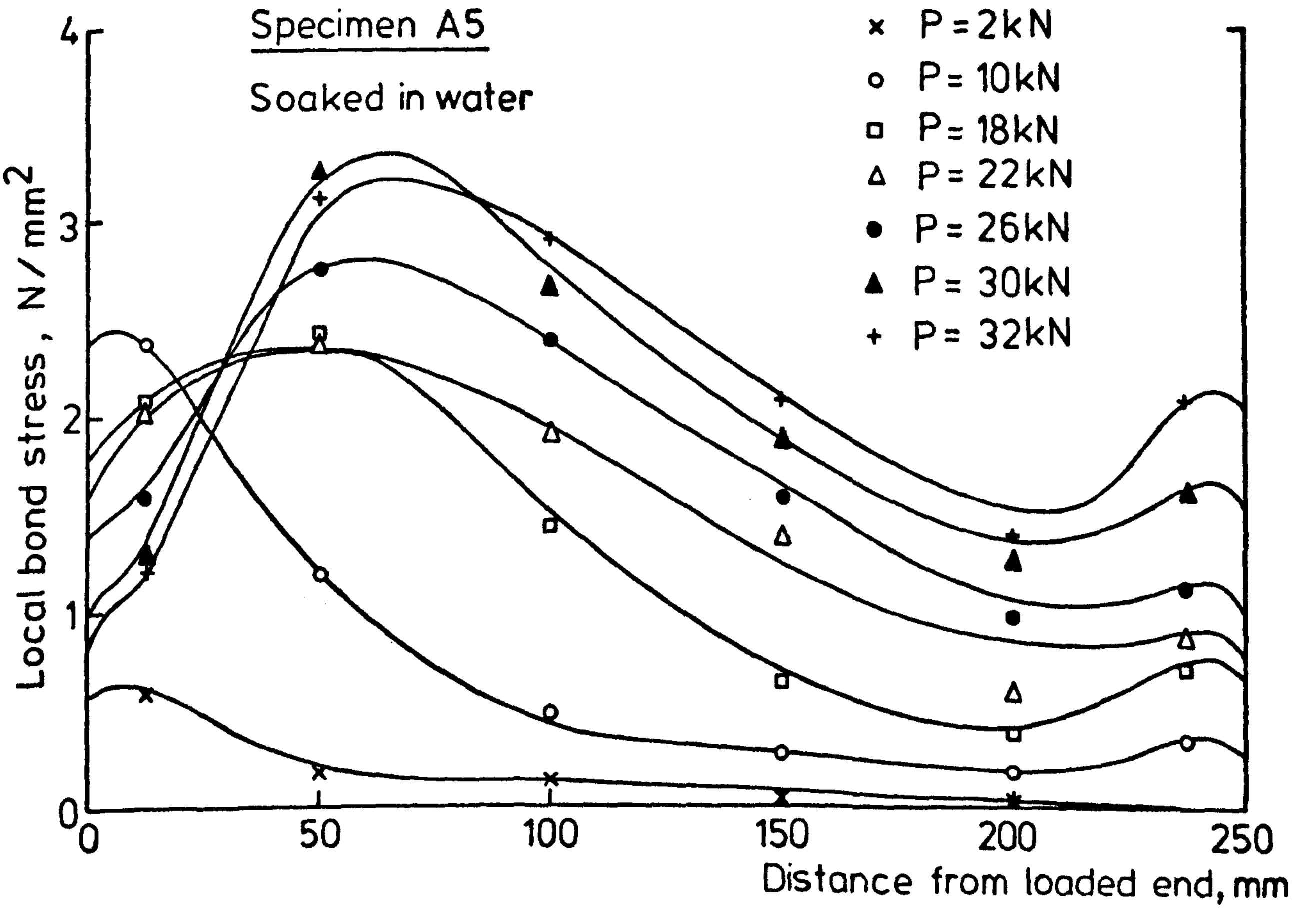


FIG. 5-20 BOND STRESS DISTRIBUTION ALONG THE BAR FOR THE WATER SATURATED SPECIMENS

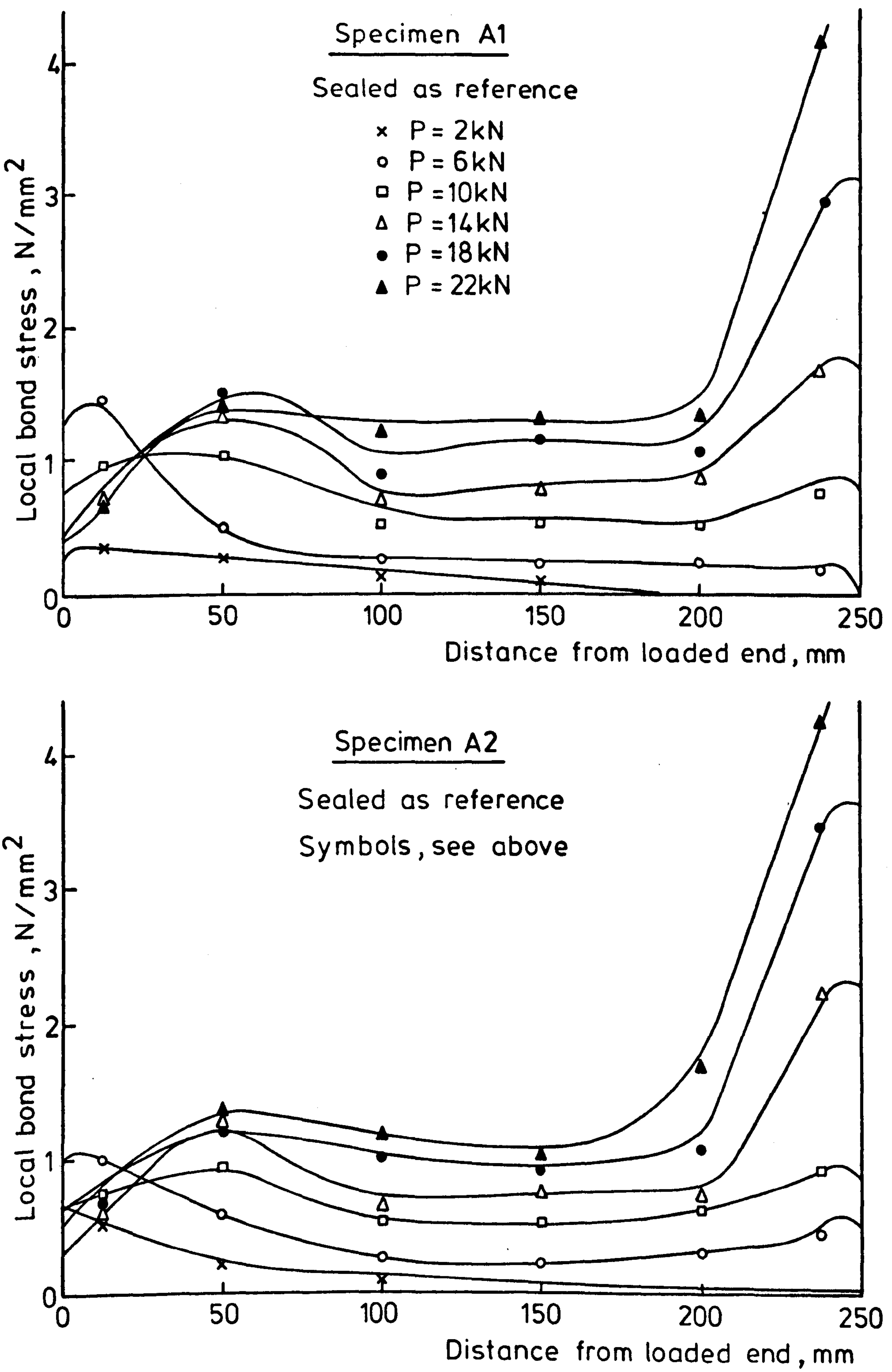


FIG. 5-21 BOND STRESS DISTRIBUTION ALONG THE BAR FOR THE SEALED SPECIMENS

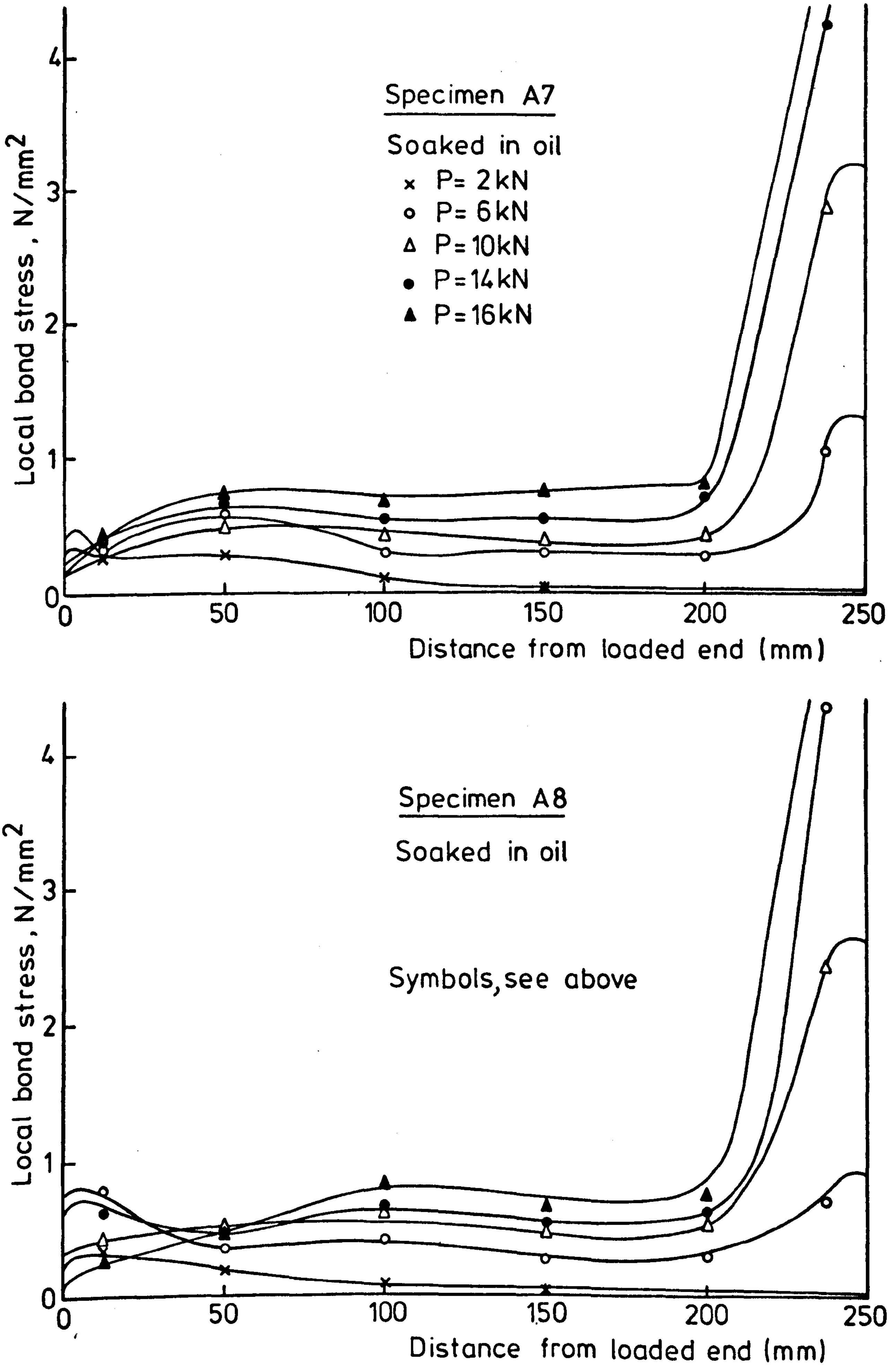


FIG. 5-22 BOND STRESS DISTRIBUTION ALONG THE BAR FOR THE OIL SATURATED SPECIMENS

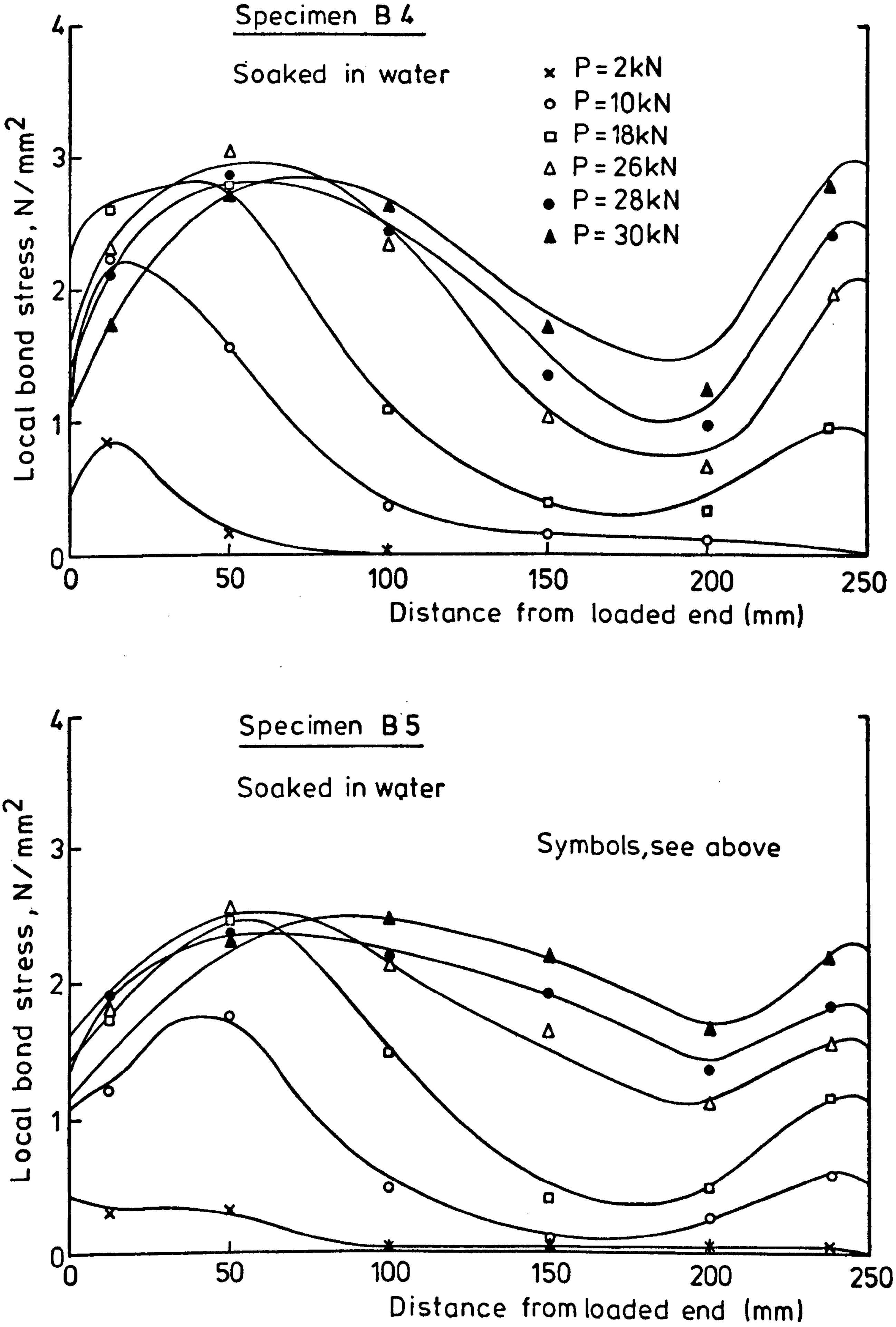


FIG. 5-23 BOND STRESS DISTRIBUTION ALONG THE BAR FOR THE WATER SATURATED SPECIMENS

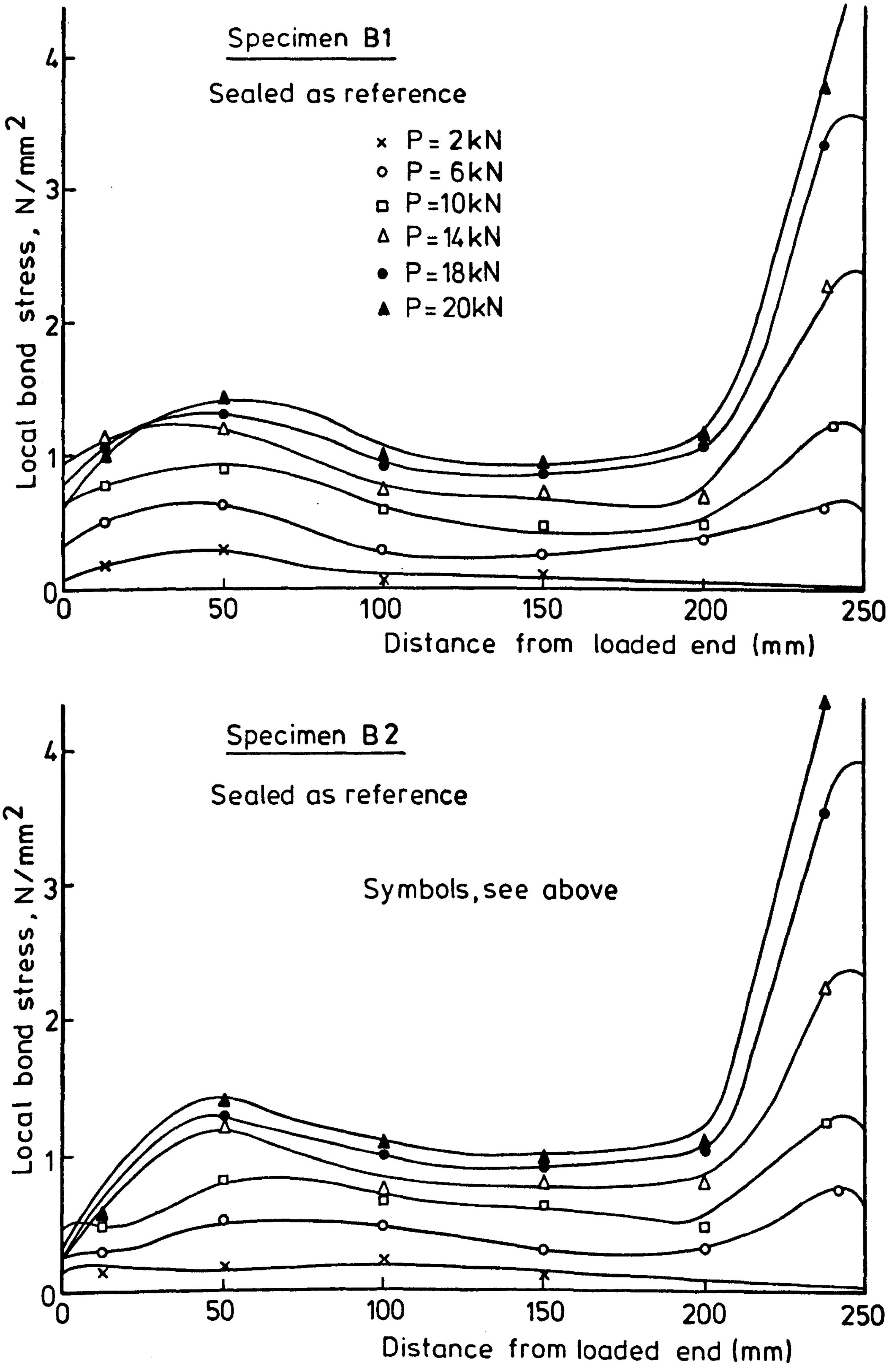


FIG. 5-24 BOND STRESS DISTRIBUTION ALONG THE BAR FOR THE SEALED SPECIMENS

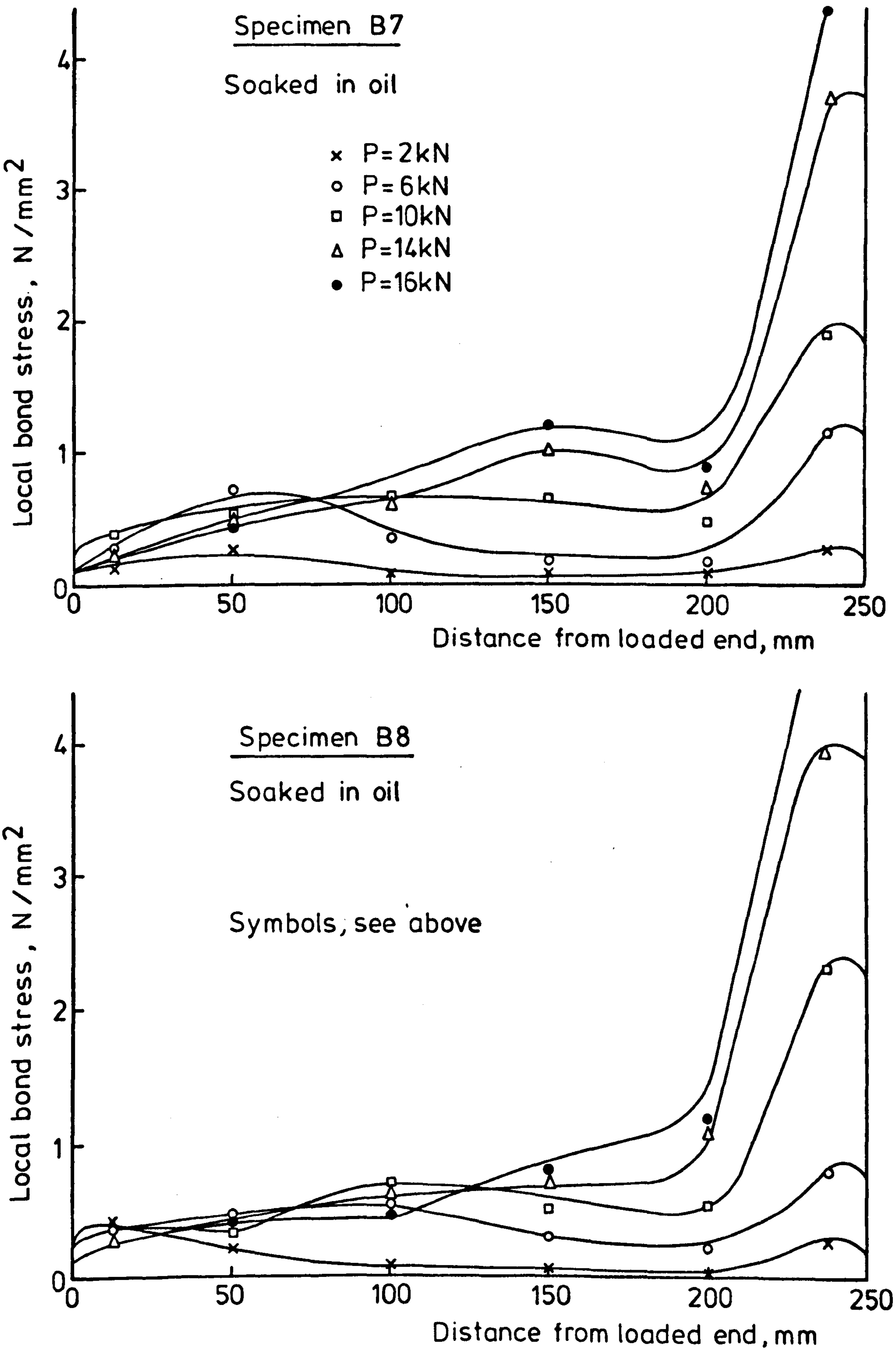


FIG. 5-25 BOND DISTRIBUTION ALONG THE BAR FOR THE OIL SATURATED SPECIMENS

findings of many investigators(117,119,121) who concluded that the maximum local bond stress occurred near the loaded end at relatively low loads and moved towards the unloaded end as the load was increased.

The bond stress of the oven dried specimens soaked in water for 8 months, started near the loaded end of the bar with a zero bond stress along the remaining bar length. As the applied load reached 10-18 kN, another increase in bond stress occurred at the embedded end of the bar. At an applied load of greater than 18 kN, about 50% of the ultimate load, the bond stress at the loaded end started to decrease accompanied by a continuous increase at the far end of the reinforcing bar. At the ultimate load, the bond stress at both ends was almost equal and greater than at any other parts of the bar. The maximum local bond stress exceeded the average bond stress by a factor of about 1.30. The reasons why higher uniform bond stress was distributed along the bar in the water cured specimens compared to those of the sealed specimens, figs. (5.20-5.25), might be for the same reasons as the change in the compressive strength of dry concrete cubes when resoaked in water for a long period, chapter 4, section (4.41).

In the case of the sealed specimens, the bond stress at the loaded end of the bar increased with the applied load up to a value of 10-14 kN. which is about 45% of the ultimate applied load. As the applied load increased further, the bond stress decreased at the loaded end and increased toward the unloaded end. At an applied load of 60% of the failure load, the local bond stress developed at the embedded end was between 1.5-2 times higher than that developed at the loaded end. At the maximum applied load, the peak bond stress was at the embedded end of the bar while the remaining part of the bar showed a lower uniform bond stress distribution. The maximum local bond stress developed at the embedded end was between 4-4.5 N/mm^2 which is about 2.5 times greater than the calculated average bond stress.

In the case of oil saturated specimens, the transfer of force to the surrounding concrete was decreasing and the stresses in the steel bar were very high. Consequently the ultimate local bond stress developed at the loaded end was between 40-60% of that for the companion sealed specimens. At the maximum applied load, the value of local bond stress at the embedded end was about six times greater than that at the loaded end. These results show clearly that at the loaded end, the friction bond is much less than that at the embedded end where it has practically maintained the maximum value. The presence of absorbed oil at the interface between the reinforcing bar and the concrete seemed to reduce the frictional resistance and pushed the rupture point towards the embedded end at a lower value of applied load.

When rupture of the adhesion occurred over a length of the bar, relative movement between the steel bar and the concrete took place(109). The change from adhesion to frictional restraint occurred at a rupture point which moved along the bar away from the loaded end. The position of the rupture point was determined from the plot of the applied load against the load at the gauge position along the bar, as shown in figs. (5.26, 5.27). When the rupture point passed any strain gauge point, the reading of that gauge should have been linear and of different slope. The gradient of this slope increased as the point considered was moved further from the loaded end. This might have been due to the fact that the frictional effects were assumed to be smaller towards the loaded end because in this region there would have been greater relative movement between the steel and concrete with consequently greater polishing action on the concrete. The slope of the line beyond the rupture point of the oil saturated specimens was the lowest due to the reduction of the frictional resistance caused by the absorbed oil.

The effect of absorbed oil on the rate of movement of the rupture point is further investigated in fig. (5.28), which shows the relationship between the amount of applied load which caused the rupture and the position of the rupture point along the instrumented bar for the specimens of the three different storage conditions. The relationship is linear for the three different storage conditions used in the present work. The rupture points along the steel bar of oil saturated specimens occurred at a lower load compared to that of the other storage condition. The rate of movement of the rupture point towards the embedded end of the bar was 32 mm/kN for the oil saturated specimens, 28 mm/kN for the sealed specimens, and 14 mm/kN for the water saturated specimens. This high rate of movement of the rupture point for the oil saturated specimens showed a lower frictional resistance and higher relative movement between the reinforcing bar and the

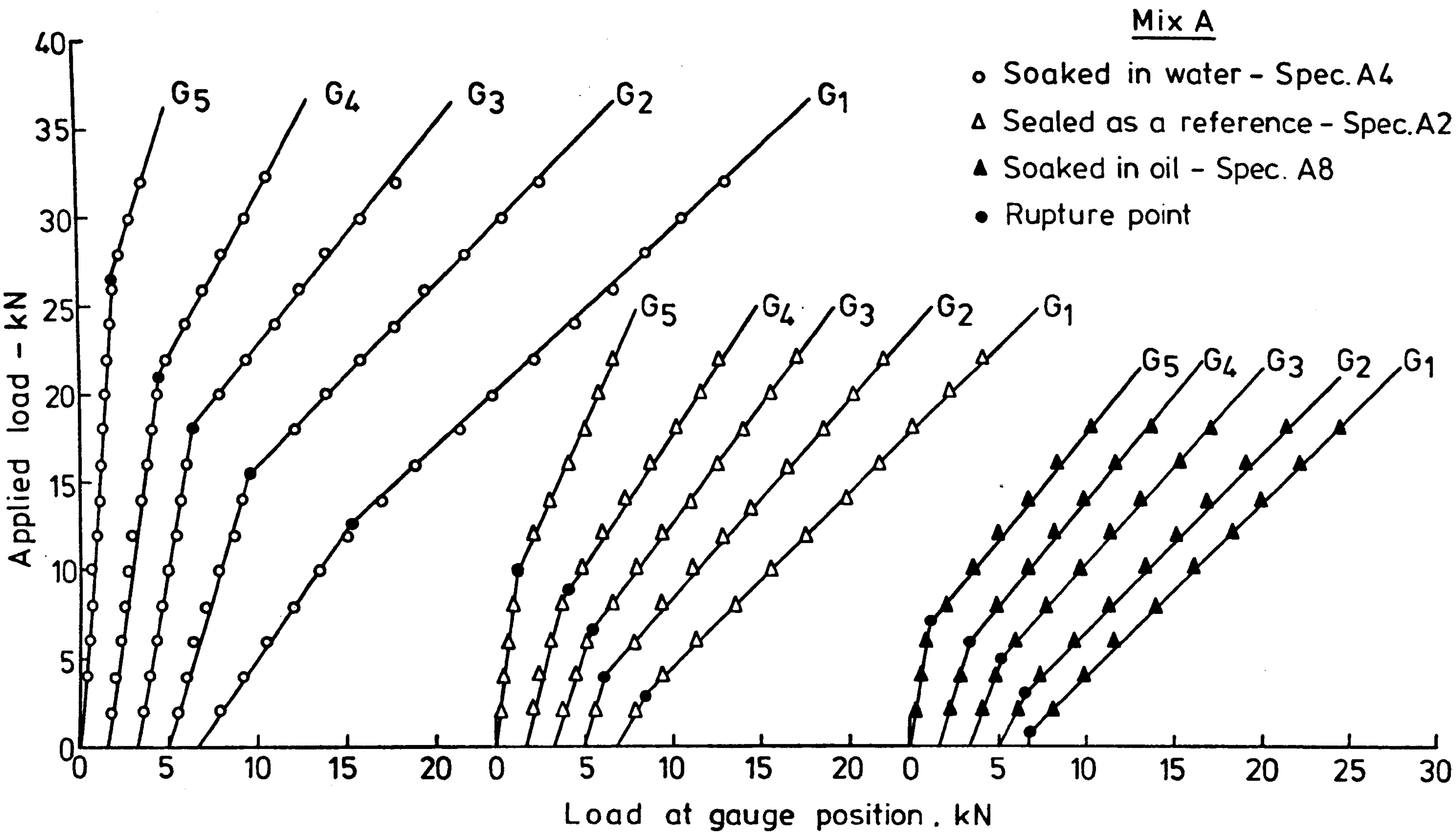


FIG. 5-26 APPLIED LOAD Vs. LOAD AT GAUGE POSITION FOR WATER SATURATED, SEALED, AND OIL SATURATED SPECIMENS

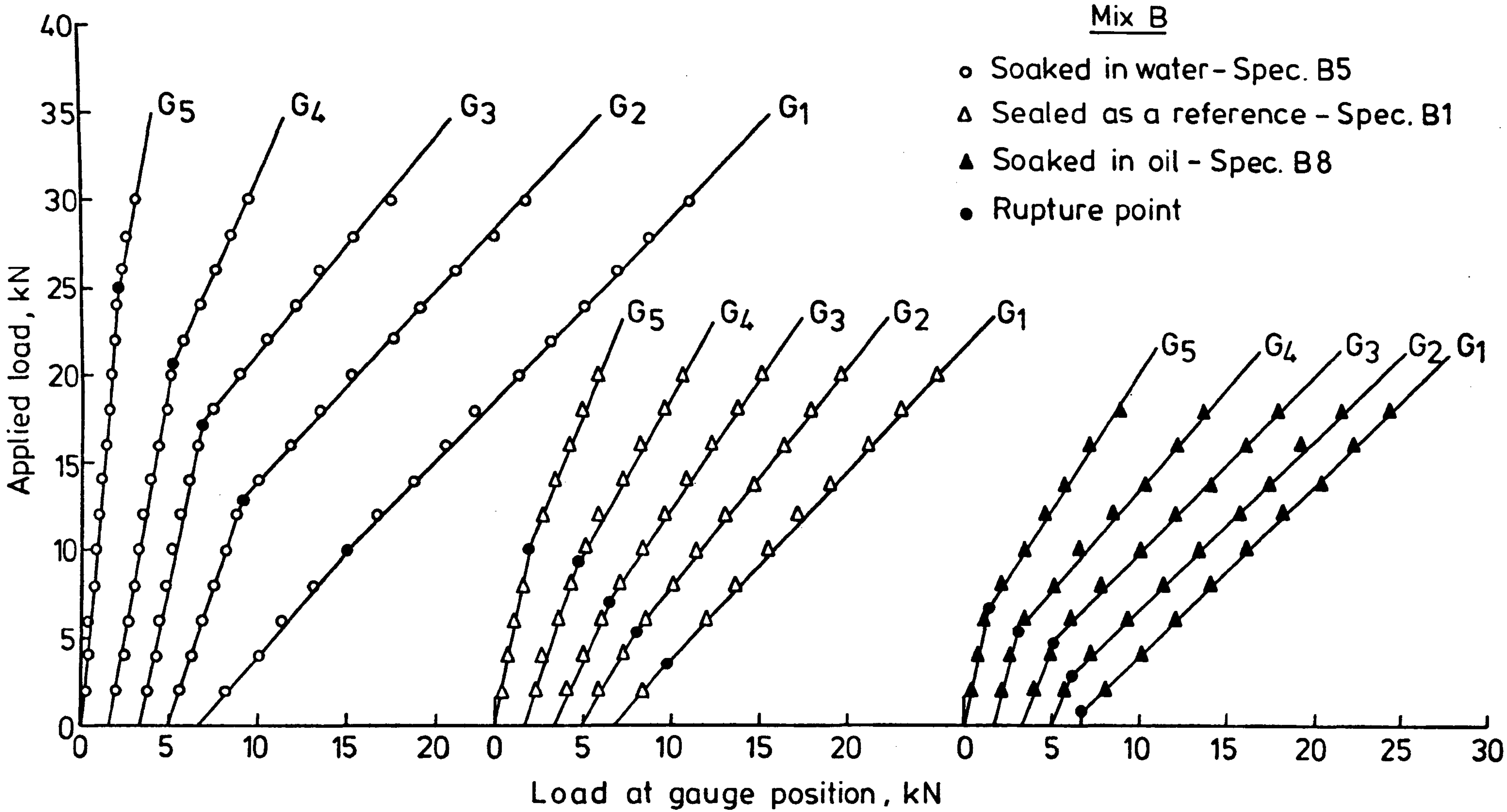


FIG. 5-27 APPLIED LOAD Vs. LOAD AT GAUGE POSITION FOR WATER SATURATED, SEALED, AND OIL SATURATED SPECIMENS

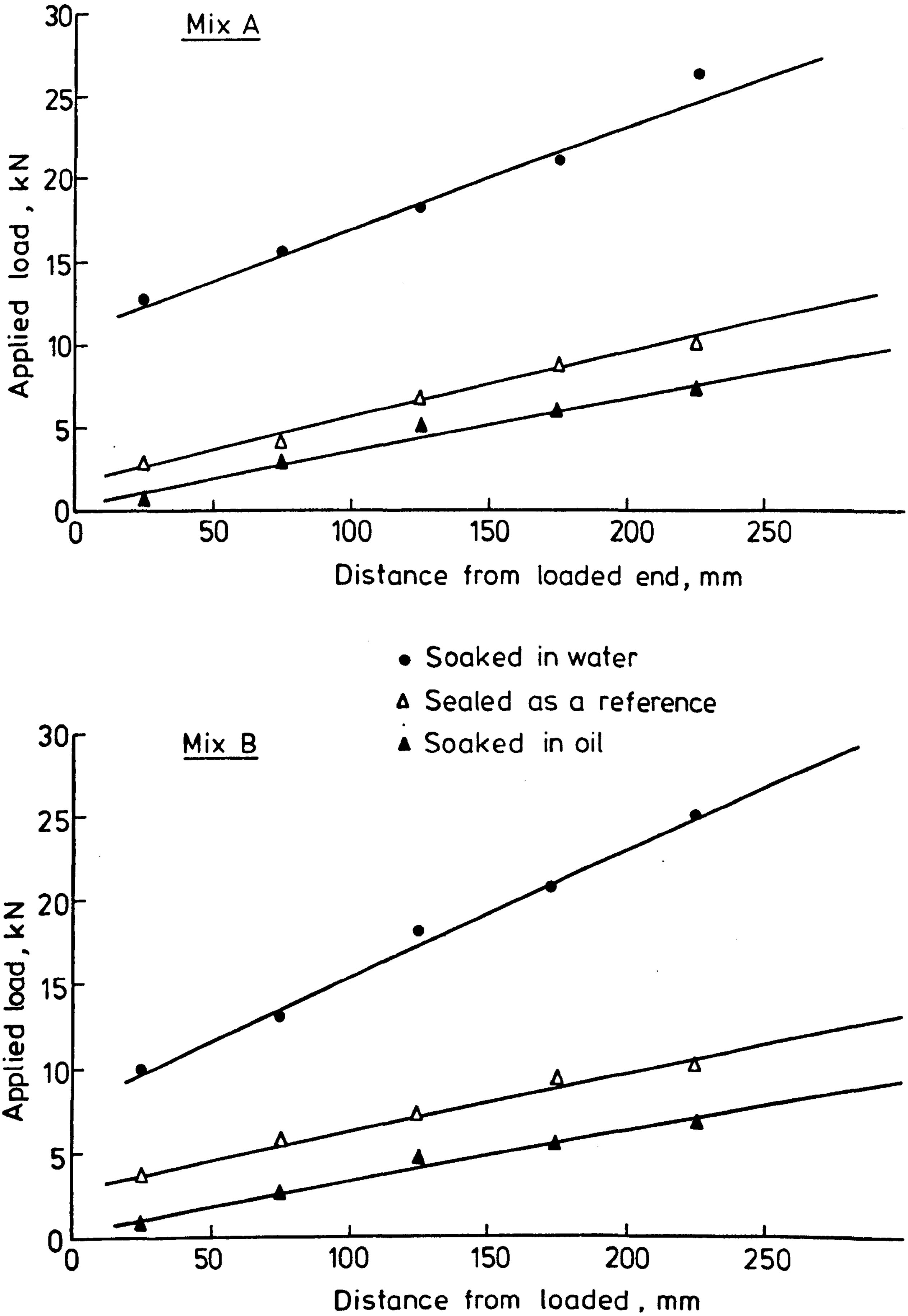


FIG. 5-28 THE FORMATION OF THE RUPTURE POINT ALONG THE REINFORCING BAR FOR THE SPECIMENS OF THE THREE DIFFERENT STORING CONDITIONS

surrounding concrete caused by the absorbed oil.

Finally, a comparison between the values of the bond stresses obtained experimentally, figs. (5.20-5.25), and those given by CP110 Code is presented in table (5.6). It can be seen that the values of the ultimate anchorage bond for water cured specimens were very near, while the values for the sealed and oil saturated specimens were lower than that given by the Code by 40% and 48% respectively. The local bond values, on the other hand, were almost equal for the water soaked specimens and much higher for the sealed and oil saturated specimens than those given by the Code. This would suggest that the usage of plain bar in reinforced concrete structures in direct contact with crude oil would cause local bond failure at low applied load compared to the normally water cured specimens.

Mix Designation	Storing condition before testing	Compressive Strength N/mm^2	Experimental		CP110 Code	
			Max. local Bond Stress	Average Bond Stress	Local Bond Stress	Average Bond Stress
A	Sealed	74.0	4.23	1.67	2.7	2.3
	Soaked in oil	72.5	6.90	1.45		
	Soaked in water	78.5	3.12	2.37		
B	Sealed	46.5	4.15	1.59	2.7	2.3
	Soaked in oil	44.0	5.83	1.39		
	Soaked in water	49.0	2.78	2.23		

Table (5.6) Comparison of the experimental results and those given by the CP110 Code.

CHAPTER 6.

DYNAMIC PROPERTIES OF OIL SATURATED CONCRETE.

6.1 INTRODUCTION.

The aim of the present work was to study the changes in the dynamic modulus of elasticity as well as the damping capacity due to soaking a dry concrete specimen in crude oil. The specimens were tested with different amounts of absorbed oil. It was stated in Chapter 4 that full saturation was nearly reached after 6 months of soaking but testing was continued for up to 2 years.

The resulting changes in the oil soaked specimens were compared with similar specimens soaked in water and others sealed in a polythene bags and stored as control specimens.

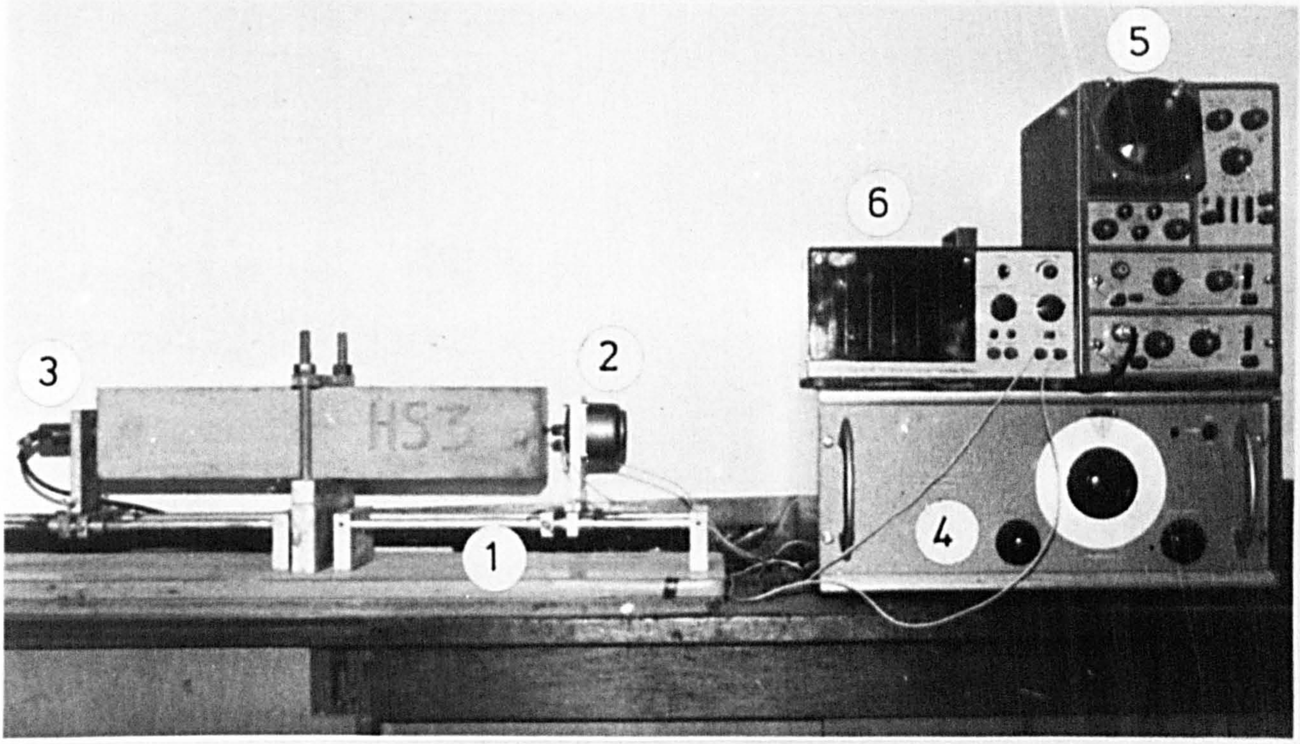
6.2 EXPERIMENTAL PROGRAMME.

The same concrete mixes (table 3.4) used for the compression test were used in this investigation. For each mix, 9 prisms (100x100x500 mm.) were cast and tested after different soaking periods. The casting, curing, and storing conditions followed the same procedure used in the compression test programme, section (4.2). The experimental programme consisted of:-

1. The measurement of the resonant frequency of concrete prisms vibrating in the longitudinal mode and the calculation of dynamic modulus as explained in section (6.4).
2. Measurement of the band width corresponding to the 50% resonant amplitude and the calculation of the logarithmic decrement as explained in section (6.4).

6.3 APPARATUS.

The experimental arrangement shown in plate (6.1) for exciting the longitudinal mode of vibration conforms with that specified in British Standards B.S. 1881, Part 5.



1. Longitudinal test bench
2. Electromagnetic vibrator
3. Pick - up device
4. A "5VA" power oscillator
5. Type "C" amplifier telex oscilloscope
6. Digital frequency counter

PLATE. 6-1. TEST SET-UP FOR FUNDAMENTAL RESONANCE IN LONGITUDINAL VIBRATION

A piezoelectric contact type crystal pick-up and an electromagnetic vibrator were used in the test and the control system consisted of a 5 VA low distortion power oscillator, with the exciter, providing a 0.91 Kgf (2 lbf) thrust with a frequency range of 5 Hz to 20 kHz. The signal from the pick-up was amplified and displayed by a Telequipment D83 Oscilloscope. The frequency was directly read on a "Racal" digital frequency counter.

In measuring dynamic properties by vibration, four basic requirements have to be satisfied:

1. The specimen must be excited in a truly sinusoidal manner and errors due to a nonsinusoidal wave-form and asymmetry of the response curve should be avoided.
2. The frequency reading should be direct and accurate, otherwise damping measurement will be inaccurate.
3. The oscillator must be stable with time, and the amplifier should have a linear response proportional to amplitude.
4. The external energy loss must be minimum at the support.

The ideal solution to satisfy the last requirement, is to suspend the specimen from fine wires. With dense materials like concrete, however, considerable energy must be imparted to the specimen to excite resonance, and suspension techniques are not always practicable. Supporting the specimen at a node is generally satisfactory.

6.4 TESTING PROCEDURE.

The concrete prism was clamped at its mid-point and the electromagnetic vibrator and pick-up unit were carefully placed in contact at the centre of each end of the specimen. Petroleum jelly was used to provide slight adhesion between the contacts and the concrete. The prisms were then excited to resonance in the fundamental longitudinal mode of vibration only. The vibration of the free end of the prism was detected by the pick-up unit and after amplification their magnitude was indicated on an oscilloscope where it was adjusted to a convenient scale. The pressure of the exciter against the specimen was adjusted to give the best resonance.

The resonance conditions were obtained by varying the frequency of the oscillator until the maximum reading was observed on the oscilloscope. The resonant frequency was recorded for later use in calculating the dynamic modulus of elasticity. The band width corresponding to 50% resonant amplitude was also recorded for later calculation of the damping capacity (logarithmic decrement). The dynamic modulus of elasticity was calculated according to the equation given by B.S. 1881. The logarithmic decrement (δ) values were calculated using the equation given in section (2.7.2).

All results given are the average of three specimens. The measurements were repeated for each specimen until three similar sets of results were obtained. The density of the tested specimens was obtained by weighing. Great care was taken to ensure that the specimens were always placed in the rig in a similar manner in order to obtain consistent results. The specimens were tested after various soaking periods and changes in density were monitored throughout these tests. After measurements were completed, all the specimens were returned to their environment immediately, and no experimental evidence was found to show any changes in the dynamic properties during the time of testing.

6.5 TEST RESULTS AND DISCUSSION.

6.5.1 Dynamic Modulus of Elasticity.

The dynamic modulus of elasticity was calculated at 7 and 21 days for water cured specimens and the results are presented in table (6.1). The dynamic modulus of the 14 days oven dried specimens, after being moist cured for 7 days, are also included in this table. Each one is the average of three prisms.

The variation in the dynamic modulus of concrete, given in table (6.1), is much smaller than that of the static modulus, table (4.13), obtained by conventional destructive methods. The greater variability in the static test results is due to the higher stress levels involved and to greater errors in testing (such as those due to the testing machine, time required to measure strains, etc.) rather than to greater variability of the specimens.

6.5.1.1 Effect of Drying on the Dynamic Modulus of Elasticity.

The dynamic modulus of the 7 day water cured concrete was reduced by 17% for the 10 mm maximum size aggregate concrete and 18.5% for the 20 mm maximum size aggregate concrete, after being dried in the oven for 14 days at 105° C. The dynamic modulus of the 14 days oven dry specimens was between 22.7-27.2% less than that of specimens continuously cured in water and tested at 21 days. The larger reduction was for the concrete specimens with larger w/c ratio and was almost independent of the maximum size of aggregate used in the concrete mixes.

This large decrease in the dynamic modulus of the oven dried specimens must be caused by the removal of water from the capillary channels and gel pores and might also be attributed to the micro-cracking and irreversible changes in the structure of the hardened cement paste on drying.

Mix Designation	w/c Ratio	Wet Density gm/cm^2	Dry Density gm/cm^2	7-day Moist-Curing				7-day Moist curing + 14 oven drying				21-day Moist-curing				% Reduction in E_D due to drying as % of	
				E_D N/mm^2	C.V.	f_r	C.V.	E_D N/mm^2	C.V.	f_r	C.V.	E_D N/mm^2	C.V.	f_r	C.V.	7-day Moist-curing	21-day Moist-curing
A1	0.4	2.410	2.277	41599	0.30	4143	0.13	34586	1.49	3897	0.90	44759	0.43	4375	0.51	16.86	22.7
A2	0.5	2.397	2.256	39162	0.89	4039	0.46	32539	1.10	3798	0.39	43253	1.27	4312	0.72	16.91	24.77
A3	0.6	2.385	2.239	36474	1.09	3910	0.54	29990	1.20	3660	0.68	41200	1.60	4222	0.83	17.78	27.21
B1	0.4	2.443	2.314	43857	1.08	4237	0.44	35818	1.69	3934	0.83	46804	1.29	4435	0.56	18.33	23.47
B2	0.5	2.427	2.294	41846	0.76	4152	0.42	33912	1.27	3845	0.50	45440	1.30	4382	0.69	18.96	25.37
B3	0.6	2.410	2.276	39370	1.14	4043	0.64	31873	1.40	3742	0.88	43882	1.55	4323	0.61	19.04	27.37

Table (6.1) The effect of composition, w/c ratio, and initial curing on the resonant frequency and the dynamic modulus of concrete prisms.

Similar findings were reported by many investigators, despite some variation in curing conditions and method of drying. David and Troxell(156) observed that the modulus of elasticity of concrete was 12-30% higher for wet concrete than dry. Serada et al.(157) while investigating the effect of absorbed water on the mechanical properties of hardened cement paste, demonstrated that no change in elastic modulus occurred between 0-50% R.H., but an increase in modulus was observed over 50% R.H. This increase, according to them, appeared to be associated with the entry of interlayer water into tobermorite gel. Johnston(158) also reported changes in the modulus of elasticity of concrete in tension for different exposure conditions and reported a reduction of 32% in the modulus value on decreasing the relative humidity from 100-30%. He, however, observed that there was no significant reduction in the modulus between 100-50% R.H. Cook(159) attributed these reductions in the modulus values to the possible influence of shrinkage induced microcracking.

6.5.1.2 The Effect of Age on the Dynamic Modulus of Elasticity of Concrete.

The variation of the dynamic modulus (E_D) with age and storage conditions for the different concrete mix proportions are shown in figs. (6.1) to (6.6) and tables (6.2, 6.3). These results show that:

1. The dynamic modulus for water soaked specimens relative to the value for the dry specimen immediately before soaking increased rapidly at first during a soaking period of 30 days, and then the rate of increase gradually decreased with age. The rapid initial increase in the dynamic modulus of the dry specimens upon soaking in water was mainly due to the high rate of

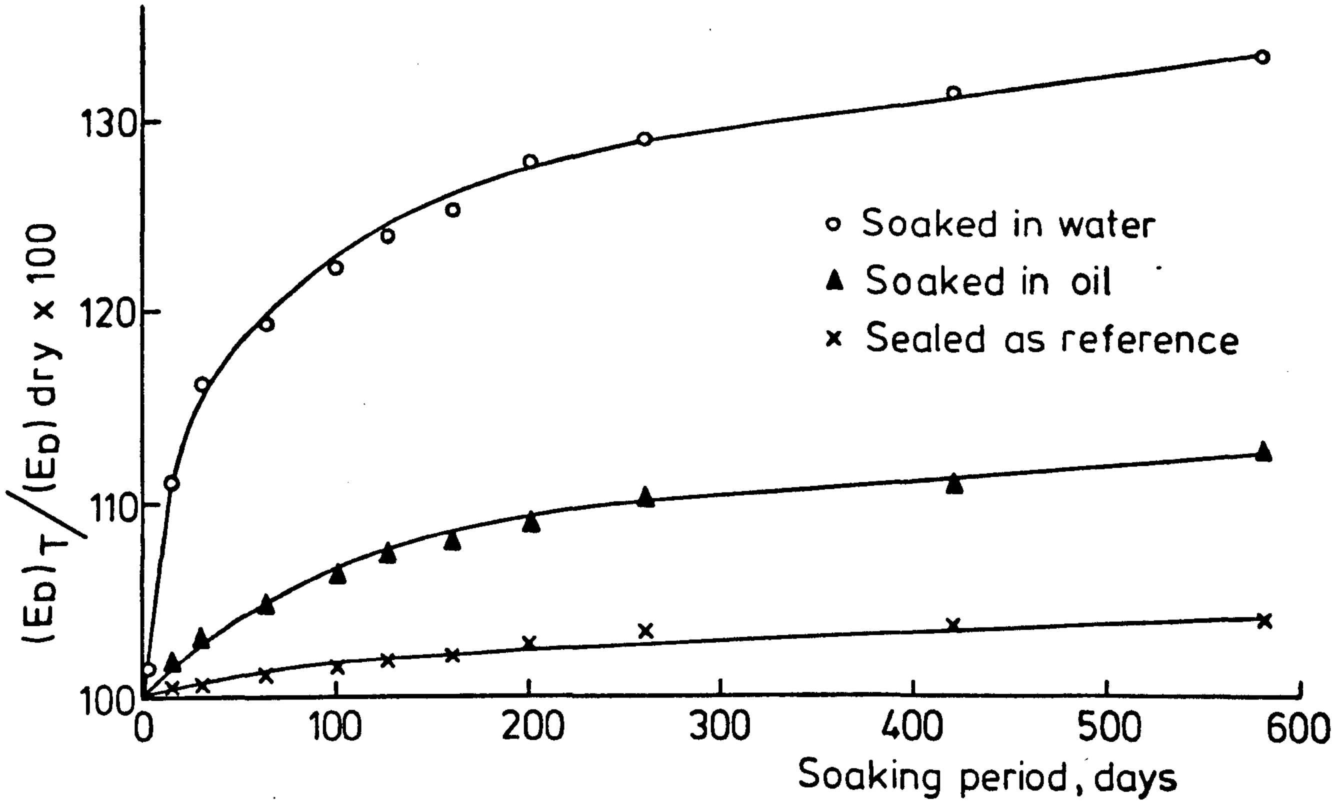


FIG. 6-1 RELATIVE CHANGE IN DYNAMIC MODULUS (E_D) OVER THAT DRIED IN OVEN FOR 14 DAYS Vs. AGE, MIX A1

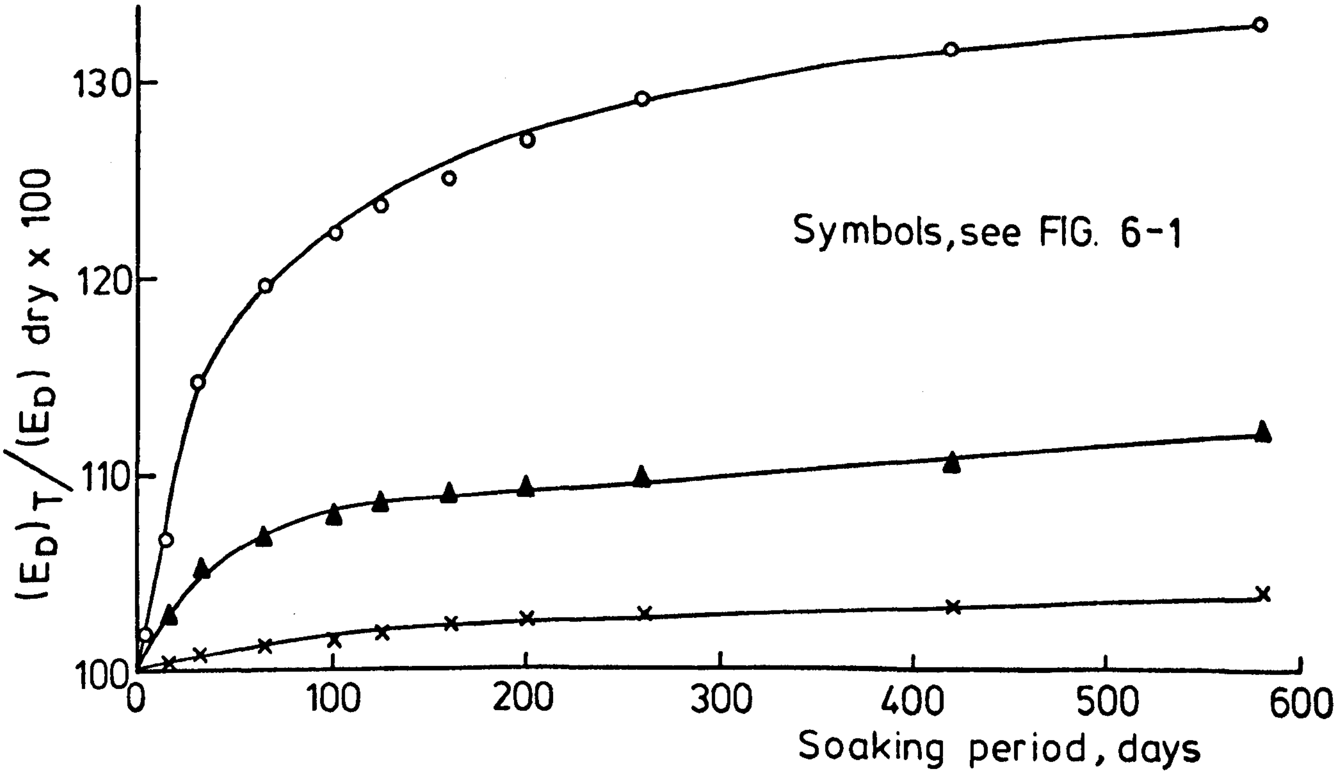


FIG. 6-2 RELATIVE CHANGE IN DYNAMIC MODULUS (E_D) OVER THAT DRIED IN OVEN FOR 14 DAYS Vs. AGE, MIX A2

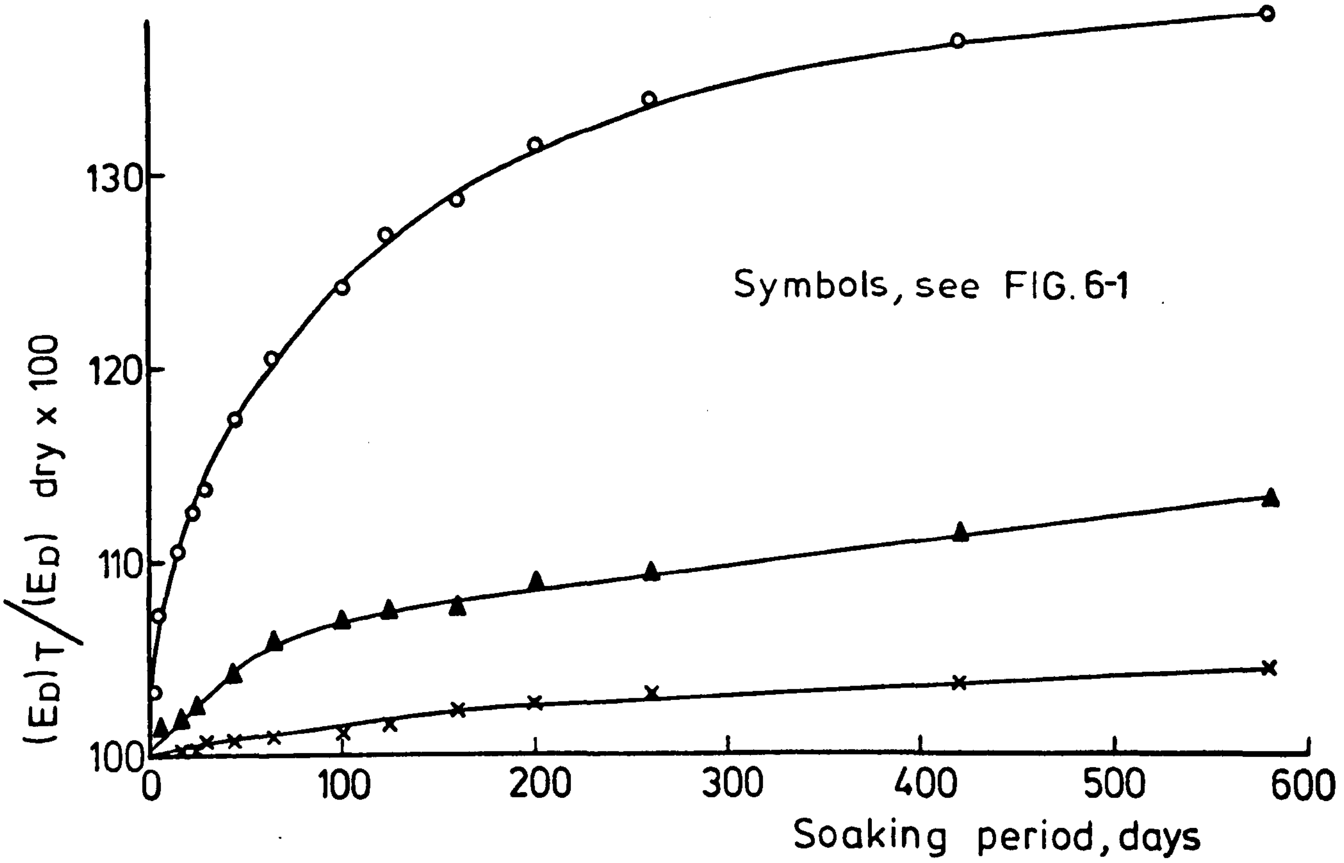


FIG. 6-3 RELATIVE CHANGE IN DYNAMIC MODULUS (E_D) OVER THAT DRIED IN OVEN FOR 14 DAYS Vs. AGE, MIX A3

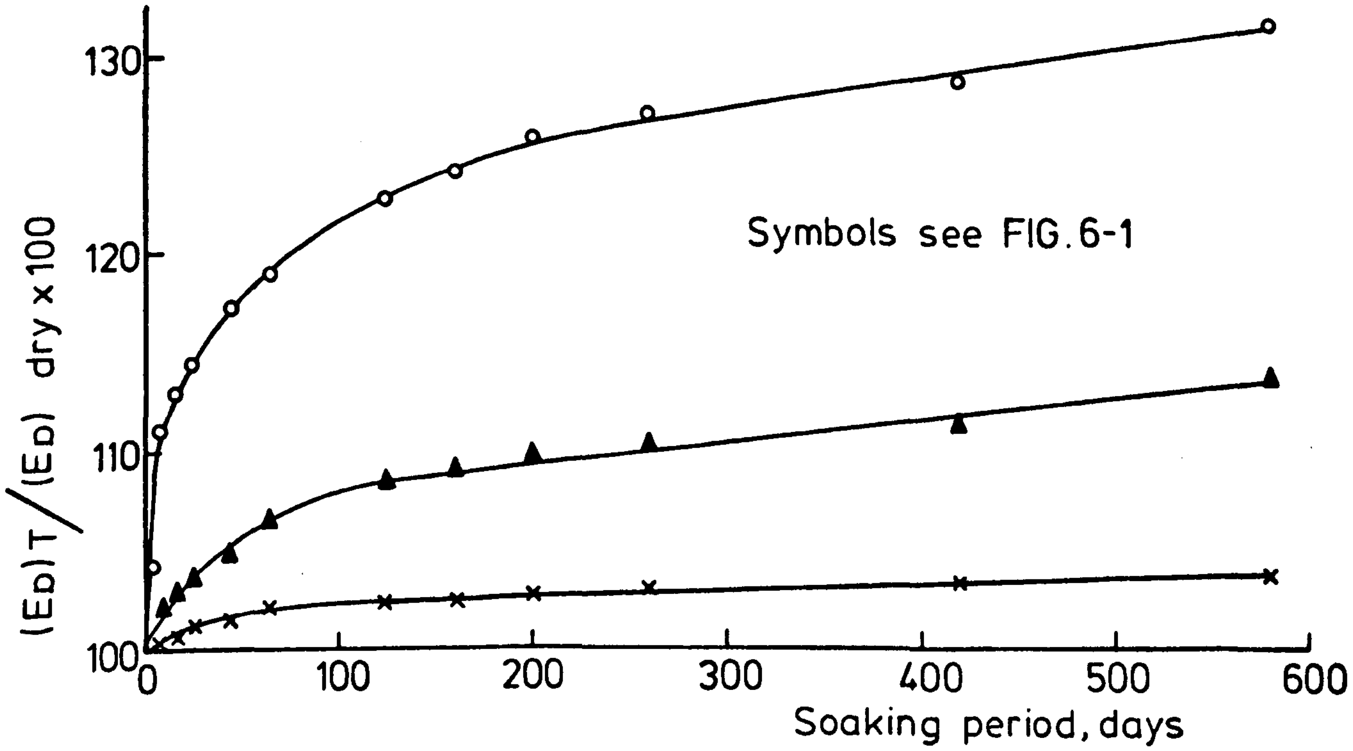


FIG. 6-4 RELATIVE CHANGE IN DYNAMIC MODULUS (E_D) OVER THAT DRIED IN OVEN FOR 14 DAYS Vs. AGE, MIX B1

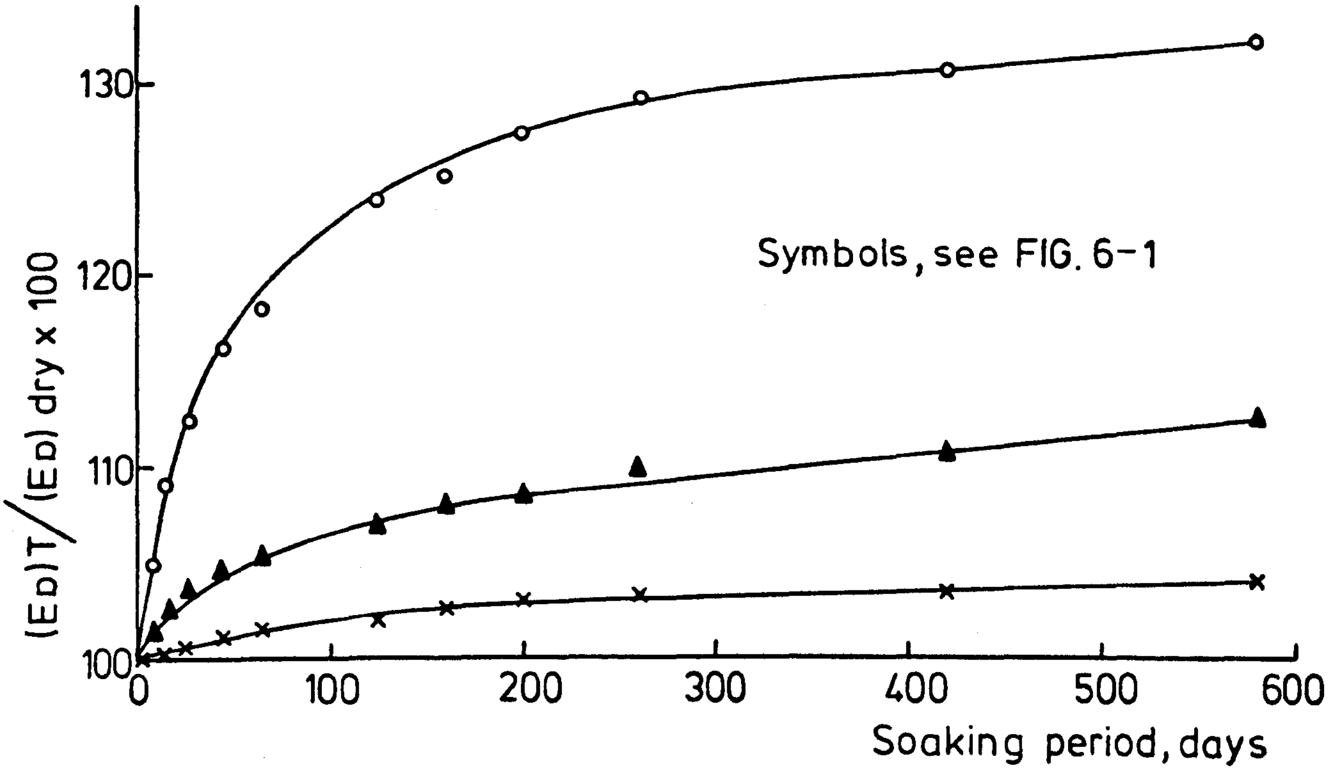


FIG. 6-5 RELATIVE CHANGE IN DYNAMIC MODULUS (Eᴅ) OVER THAT DRIED IN OVEN FOR 14 DAYS Vs. AGE, MIX B2

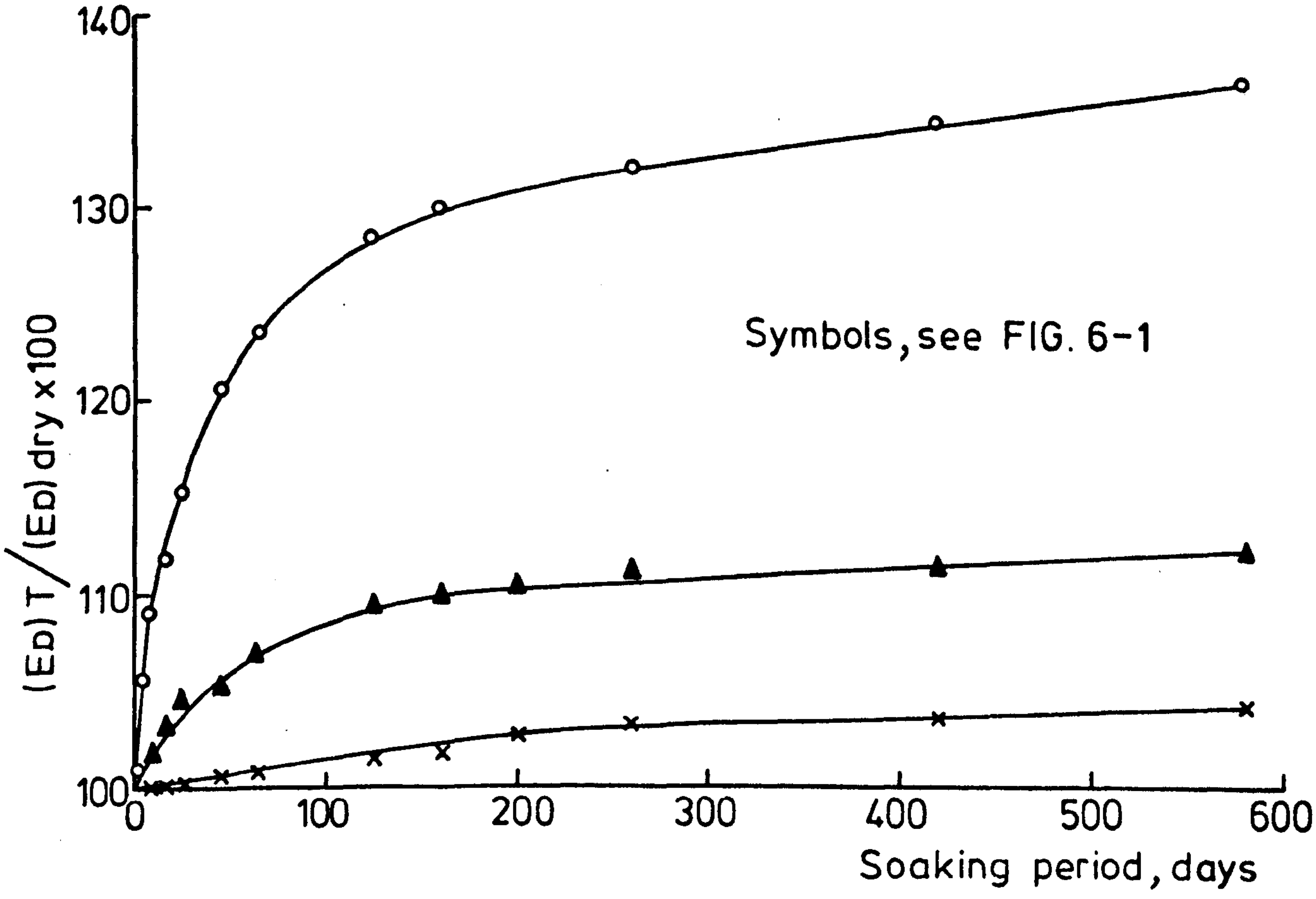


FIG. 6-6 RELATIVE CHANGE IN DYNAMIC MODULUS (Eᴅ) OVER THAT DRIED IN OVEN FOR 14 DAYS Vs. AGE, MIX B3

	A_1						A_2						A_3					
	Sealed		Soaked in Oil		Soaked in water		Sealed		Soaked in oil		Soaked in water		Sealed		Soaked in oil		Soaked in water	
	Dynamic Modulus (E_D)	Density (ρ)	Dynamic Modulus (E_D)	Density (ρ)	Dynamic Modulus (E_D)	Density (ρ)	Dynamic Modulus (E_D)	Density (ρ)	Dynamic Modulus (E_D)	Density (ρ)	Dynamic Modulus (E_D)	Density (ρ)	Dynamic Modulus (E_D)	Density (ρ)	Dynamic Modulus (E_D)	Density (ρ)	Dynamic Modulus (E_D)	Density (ρ)
7-days Moist Curing	120.2	105.86	120.2	105.86	120.2	105.86	120.4	106.26	120.4	106.26	120.4	106.26	121.6	106.54	121.6	106.54	121.6	106.54
14-days oven drying	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100
1	99.80	100.01	99.56	100.54	99.22	105.07	99.57	100.04	100.24	100.56	99.27	105.34	99.82	140.09	100.49	100.60	99.06	106.31
3	100.12	100.67	100.23	100.64	101.40	105.25	99.78	100.04	100.77	100.87	101.79	105.76	99.88	100.10	100.84	101.22	103.16	106.40
7	100.20	100.10	100.86	101.06	105.06	105.33	100.17	100-26	101.84	101.74	103.92	106.03	99.94	100.16	101.33	102.33	107.15	106.45
15	100.40	100.25	101.91	108.02	111.09	105.60	100.42	100.11	102.82	102.44	106.61	106.16	100.30	100.19	101.74	102.40	110.52	106.50
25	100.48	100.29	102.52	102.26	114.15	105.69	100.53	100.16	104.59	103.09	110.57	106.27	100.36	100.20	102.46	102.97	112.73	106.59
30	100.62	100.32	103.11	102.69	116.33	105.73	100.81	100.23	105.31	103.34	114.52	106.30	100.58	100.28	103.68	103.56	113.97	106.62
45	100.76	100.35	103.89	103.25	116.83	105.75	101.00	100.26	106.18	103.71	116.24	106.33	100.67	100.29	104.31	103.86	117.33	106.65
65	101.04	100.42	104.57	103.61	119.35	105.85	101.13	100.34	106.74	103.93	119.59	106.43	100.91	100.36	106.01	104.18	120.44	106.78
100	101.50	100.62	106.20	103.78	122.50	105.90	101.50	100.38	107.76	104.33	122.12	106.50	100.97	100.37	106.89	104.70	124.31	106.83
125	101.71	100.93	107.40	104.20	123.80	105.93	101.88	100.50	108.47	104.54	123.65	106.52	101.56	100.51	107.38	104.90	127.03	106.39
160	101.96	100.82	108.08	104.41	125.30	106.00	102.31	100.56	108.90	104.61	124.96	106.66	102.20	100.60	107.48	104.94	128.67	106.92
200	102.64	100.87	108.85	104.67	127.66	106.09	102.39	100.58	109.25	104.68	127.04	106.71	102.46	100.70	108.89	105.01	131.44	106.94
260	103.16	101.03	110.14	104.80	128.88	106.15	102.75	100.73	109.71	104.80	128.97	106.73	102.96	100.75	109.45	105.21	133.97	107.04
420	103.65	101.04	110.80	104.96	131.39	106.28	103.27	100.92	110.47	104.98	131.60	106.75	103.69	100.86	111.47	105.28	136.87	107.13
580	104.07	101.09	112.73	105.10	133.18	106.39	104.04	100.93	112.15	105.22	133.20	106.81	104.37	100.92	113.25	105.50	138.42	107.17

Table (6.2) Relative change of the dynamic modulus and density with age to that of the oven dried samples - Mix A.

Soaking period (days)	B_1						B_2						B_3					
	Sealed		Soaked in oil		Soaked in water		Sealed		Soaked in oil		Soaked in water		Sealed		Soaked in oil		Soaked in water	
	Dynamic Modulus (E_D)	Density (ρ)	Dynamic Modulus (E_D)	Density (ρ)	Dynamic Modulus (E_D)	Density (ρ)	Dynamic Modulus (E_D)	Density (ρ)	Dynamic Modulus (E_D)	Density (ρ)	Dynamic Modulus (E_D)	Density (ρ)	Dynamic Modulus (E_D)	Density (ρ)	Dynamic Modulus (E_D)	Density (ρ)	Dynamic Modulus (E_D)	Density (ρ)
7-days Moist-Curing	122.4	105.6	122.4	105.6	122.4	105.6	123.4	105.8	123.4	105.8	123.4	105.8	123.5	106.1	123.5	106.1	123.5	106.1
14-days oven drying	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100
1	99.97	100.02	100.55	100.45	98.98	105.19	99.76	100.02	100.71	100.50	100.13	105.61	100.02	100.03	100.75	100.70	100.66	106.03
3	100.05	100.05	100.62	100.62	104.34	105.46	99.86	100.07	101.00	101.0	102.19	105.74	99.92	100.08	101.23	101.50	105.55	106.12
7	100.19	100.09	102.56	101.22	111.16	105.66	100.15	100.10	101.57	101.90	105.08	105.90	99.76	100.09	101.84	102.01	109.14	106.14
15	100.78	100.16	102.84	101.91	113.07	105.75	100.24	100.14	102.69	102.27	109.28	105.95	100.01	100.17	103.02	102.86	111.85	106.21
25	101.17	100.25	103.66	102.30	114.41	105.80	100.76	100.24	103.78	103.03	112.38	106.05	100.26	100.20	104.41	103.29	115.21	106.26
45	101.41	100.34	104.99	102.80	117.23	105.90	101.20	100.27	104.65	103.20	116.30	106.08	100.62	100.30	105.11	103.39	120.48	106.33
65	101.92	100.38	106.57	103.44	119.08	105.93	101.51	100.30	105.38	103.60	118.30	106.10	100.78	100.39	106.83	103.89	123.41	106.41
125	102.11	100.52	108.34	103.85	122.81	106.03	101.98	100.34	107.04	104.06	123.50	106.18	101.36	100.54	109.36	104.15	128.07	106.50
160	102.24	100.55	109.06	104.18	124.12	106.14	102.54	100.53	107.98	104.23	125.09	106.25	101.63	100.60	109.98	104.44	129.55	106.58
200	102.60	100.69	109.75	104.22	125.85	106.22	102.95	100.67	108.66	104.36	127.23	106.40	102.62	100.70	110.40	104.57	130.26	106.64
260	102.91	100.74	110.36	104.25	126.98	106.28	103.14	100.76	109.78	104.44	129.03	106.43	102.84	100.72	111.00	104.64	131.85	106.68
420	103.09	100.77	111.50	104.36	128.65	106.34	103.29	100.80	110.66	104.56	130.46	106.50	103.37	100.76	111.47	104.77	131.20	106.75
580	103.57	100.83	113.77	104.54	131.65	106.41	103.86	100.82	112.48	104.65	132.93	106.60	104.01	100.78	112.08	105.01	136.30	106.78

Table (6.3) Relative changes in the dynamic modulus and density with age to that of the oven dried samples - Mix B.

water absorption and refilling of the emptied pores which increased both the resonant frequency and the density of the concrete. The refilling of the emptied pores with water would give resumed hydration which according to Powers(28,30) results in further increase in the gel/space ratio and consequently the strength and modulus of elasticity of concrete.

2. The results presented in section (6.5.2) showed that the dynamic modulus of the 7-day water cured concrete reduced by 17-18.5% due to oven drying, but upon resaturation in water, the reduction in the dynamic modulus was completely recovered after 65 days. After a water soaking period of 580 days, the dynamic modulus had increased by 33-39% for concrete of Mix A and 31-56% for concrete of Mix B, above the dry dynamic modulus before soaking in water, and was between 8-10% higher than that of the 7-day water cured specimens before oven drying. A possible explanation for this increase in the dynamic modulus upon re-soaking in water is that the entry of interlayer water into the tobermorite gel caused a resumption of hydration which healed some of the cracks induced by drying shrinkage.

3. The dynamic modulus of oil soaked concrete specimens increased gradually during the soaking period and oil penetration. After soaking for 580 days, the increase in the dynamic modulus of the oil soaked specimens was between 12-14% greater than that of the dry specimens before soaking in oil. The percentage increase was almost the same for all concrete mixes regardless of the maximum size of aggregates and water/cement ratios. The effect of absorbed oil on the dynamic modulus is discussed later in section (6.5.1.3).

4. The dry specimens sealed in polythene bags as a reference had an increase in the dynamic modulus between 3.5-4.5% after 580 days. At the age of 580 days, the difference between the dynamic modulus of the sealed specimens and those soaked in water varied between 28-33% for the 10 mm maximum size aggregate concrete and 27-31% for the 20 mm maximum size aggregate concrete, depending upon the water-cement ratio. A great part of the difference between the dynamic modulus of the sealed and water soaked specimens, was due to the actual differences in the moisture content of concrete, as clearly shown in figs. (6.7) to (6.12) which is a plot of the change in weight during the soaking period.

6.5.1.3 Effect of Absorbed Oil on the Dynamic Modulus of Elasticity.

The dynamic modulus of the 580-days oil saturated specimens was between 8-10% higher than that of similar sealed specimens tested at the same age. The relative increase in the dynamic modulus after soaking in crude oil for 580 days was 60-65% less than the increase upon resaturating similar specimens with water for the same period. The differences could be due to the following:

1. The specimens soaked in water reached full saturation after 25 days; but those soaked with crude oil reached only 93.5-97.5% saturation for concrete of Mix A and 90-93% saturation for concrete of Mix B, after 580 days. Therefore, some pores remained empty because of the pore structure characteristics and impermeability of these pores to crude oil molecules as discussed earlier in section (4.5.5). The presence of these empty pores in the case of oil soaked specimens will reduce both the resonant frequency and the density of the concrete, thereby reducing the dynamic modulus of elasticity.

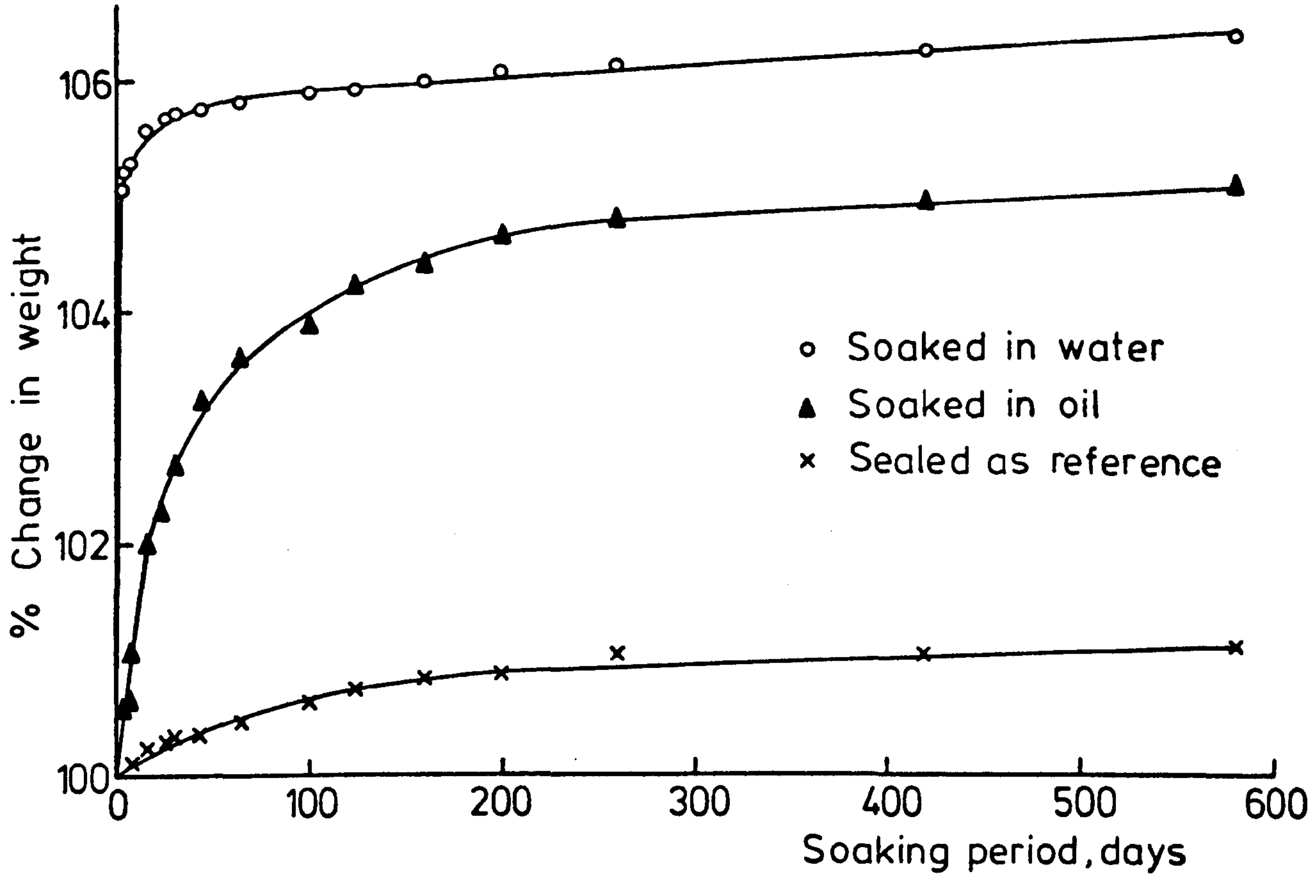


FIG. 6-7 PERCENTAGE CHANGE IN WEIGHT OF CONCRETE PRISMS Vs. AGE, MIX A1

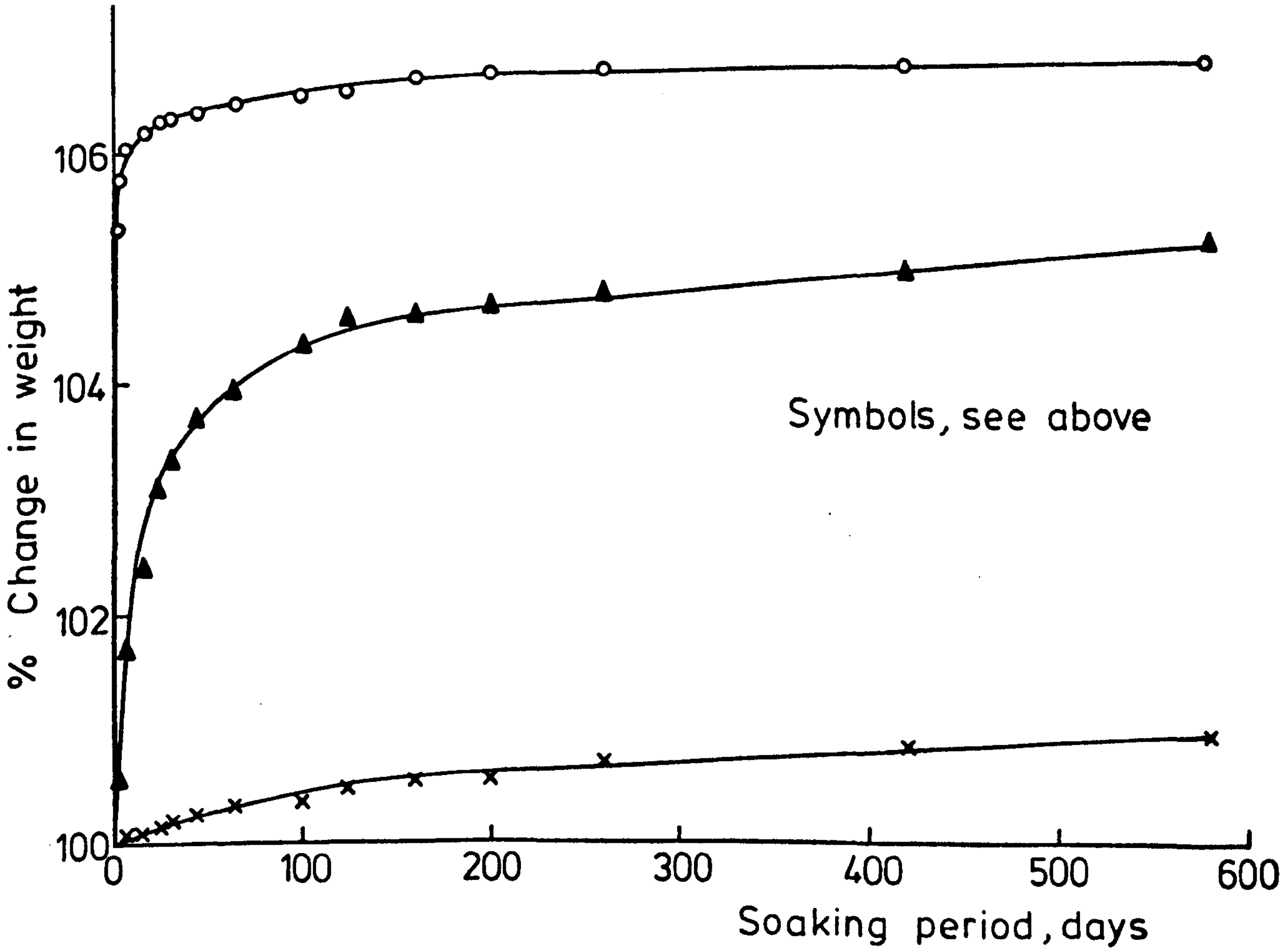


FIG. 6-8 PERCENTAGE CHANGE IN WEIGHT OF CONCRETE PRISMS Vs. AGE, MIX A2

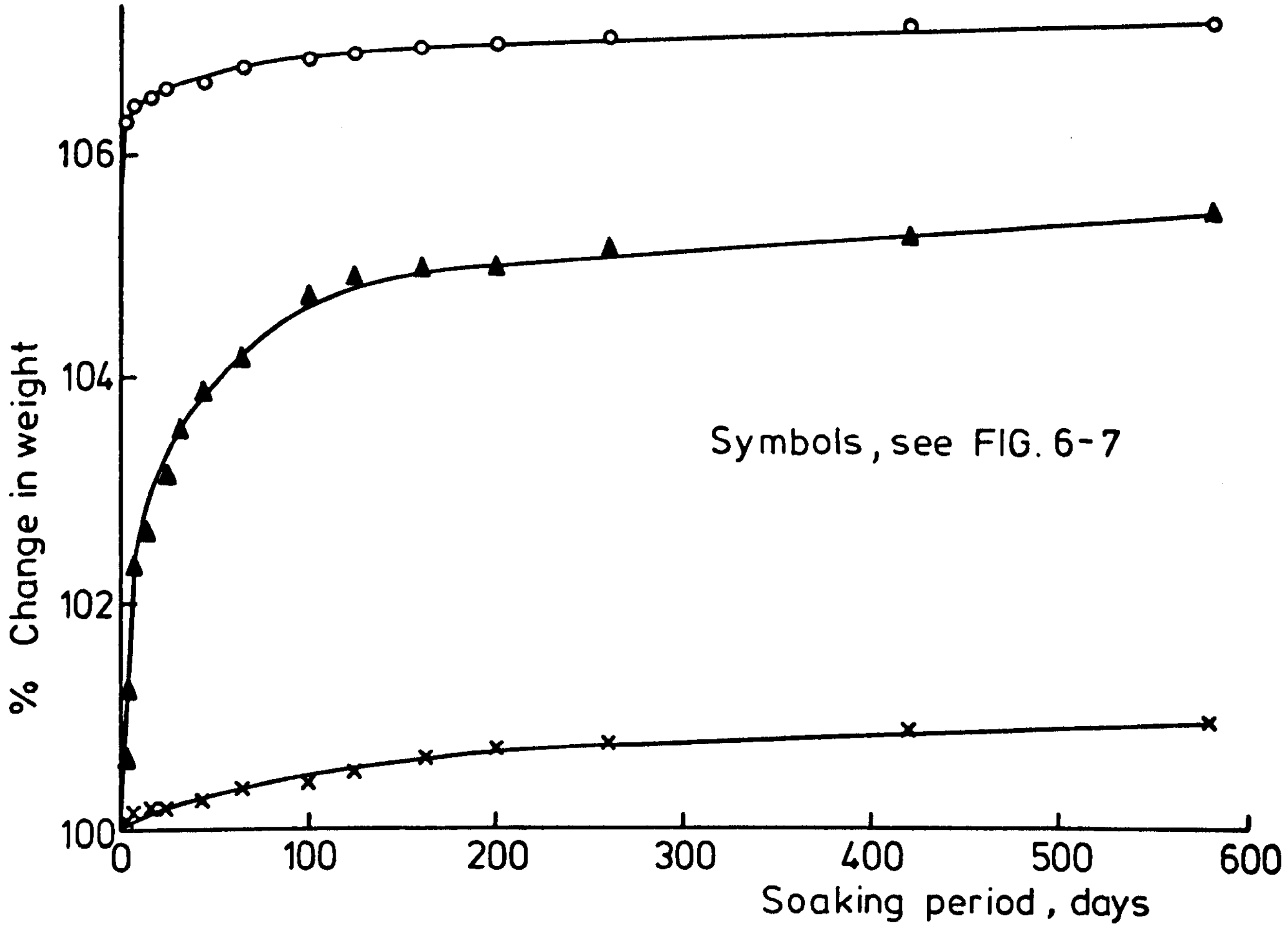


FIG. 6-9 PERCENTAGE CHANGE IN WEIGHT OF CONCRETE PRISMS Vs. AGE, MIX A3

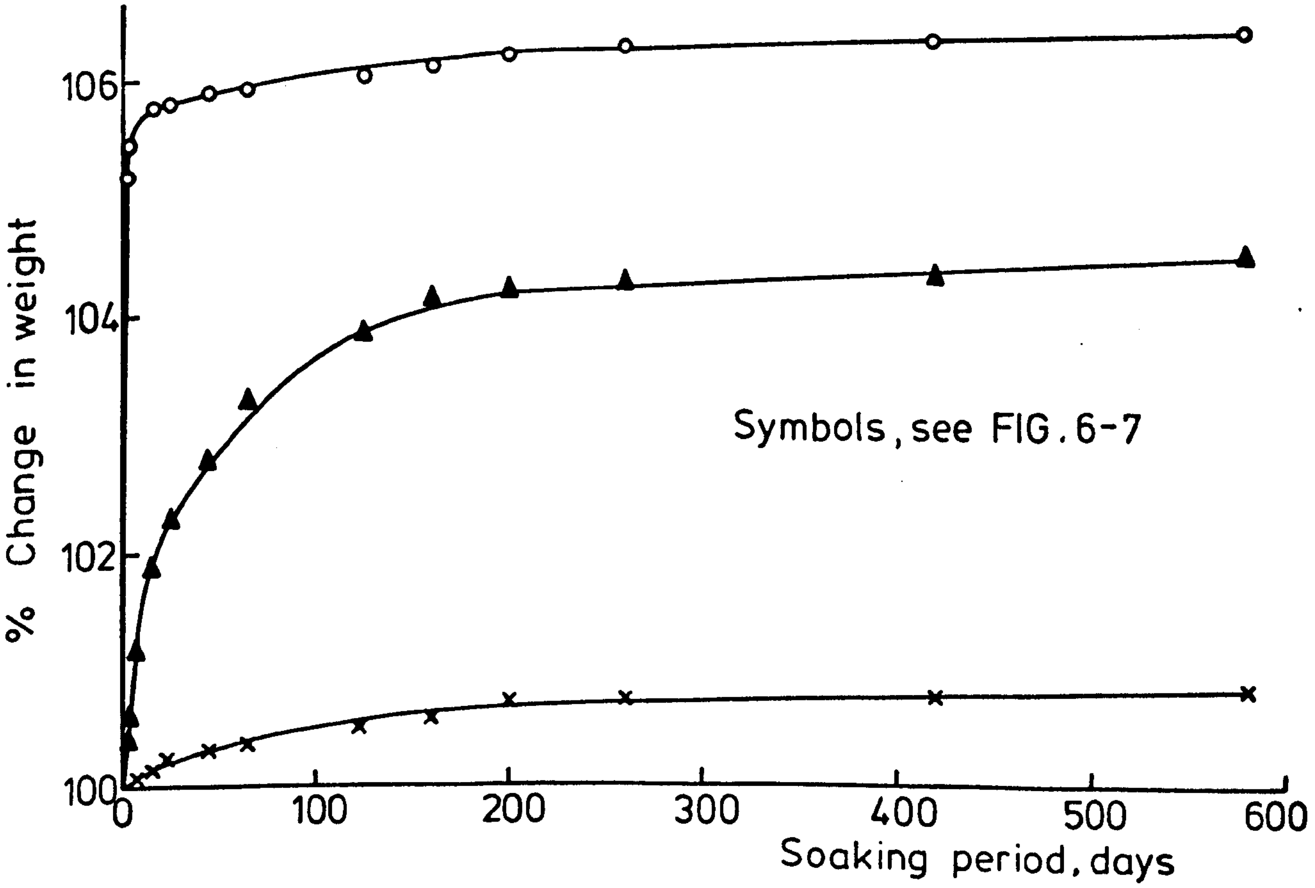


FIG. 6-10 PERCENTAGE CHANGE IN WEIGHT OF CONCRETE PRISMS Vs. AGE, MIX B1

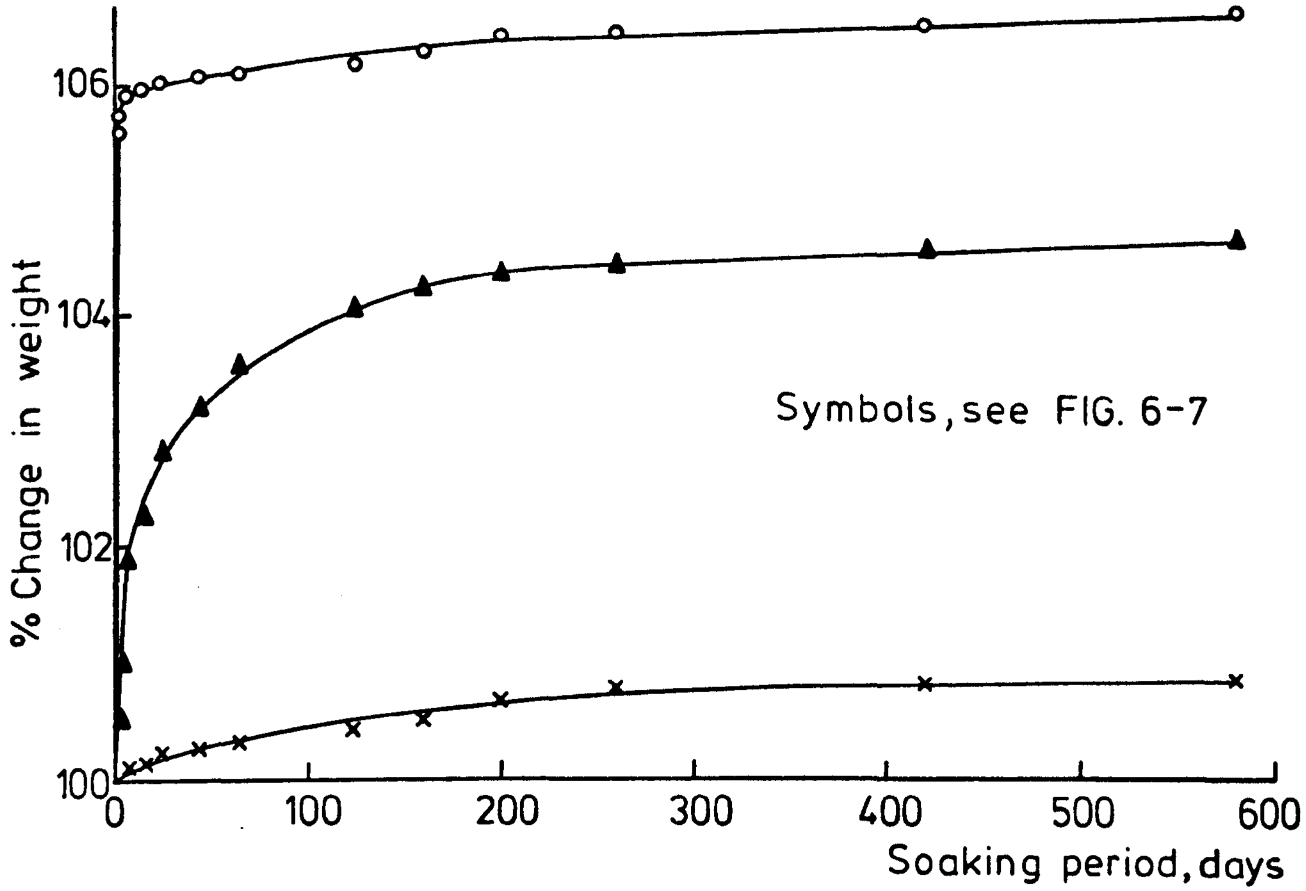


FIG. 6-11 PERCENTAGE CHANGE IN WEIGHT OF CONCRETE PRISMS Vs AGE, MIX B2

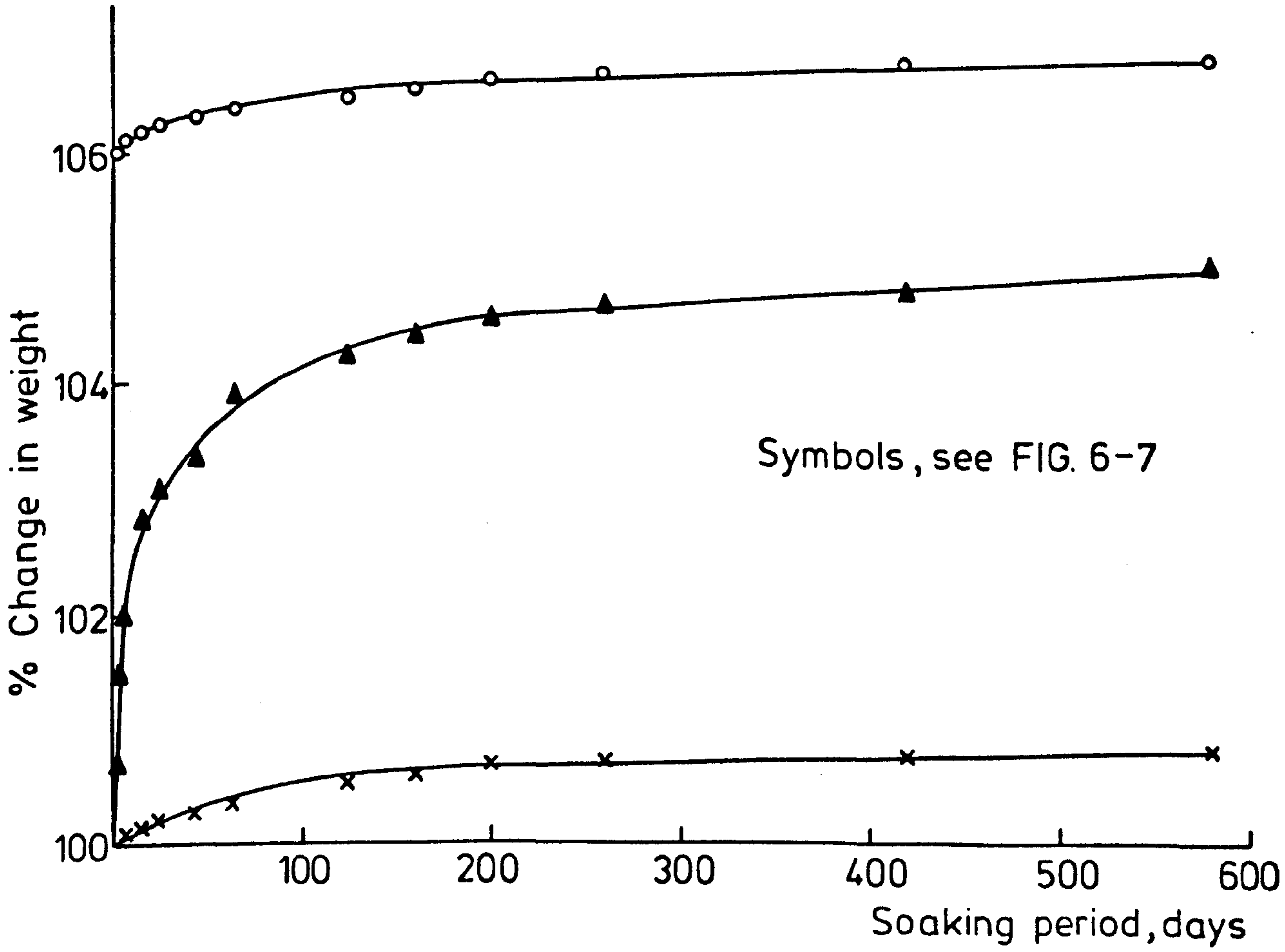


FIG. 6-12 PERCENTAGE CHANGE IN WEIGHT OF CONCRETE PRISMS Vs. AGE, MIX B3

2. Crude oil has a specific gravity of (0.88) which is less than that of water, and so oil soaked concrete has a smaller increase in weight or density than water soaked concrete. Consequently the dynamic modulus is also less.

3. Crude oil does not allow any hydration to take place in concrete. Resaturation with water, on the other hand, permits the hydration to continue and produce an increase in density and resonant frequency of concrete, thus increasing the dynamic modulus(160).

6.5.1.4 The Effect of W/C Ratio on the Dynamic Modulus of Concrete.

The effect of water-cement ratio on the dynamic modulus of the 10 mm and 20 mm maximum size aggregate concrete is shown in figures (6.13) and (6.14) respectively. These figures indicate:

1. The dynamic modulus of concrete decreased as the w/c ratio increased; for example, the dynamic modulus of the 21-days water cured concrete decreased by 6-7.5% upon increasing the water-cement ratio from 0.4 to 0.6. This might be due to the fact that the dynamic modulus of elasticity of concrete is a function of the elastic properties and volume concentration of its constituents, which may be conveniently divided into a cement paste and aggregate. Accordingly, for the same aggregate/cement ratio, the higher the water/cement ratio, the lower is the dynamic modulus, because of the increasing number and size of pores in the concrete. On the other hand, the greater the percentage of aggregate, the higher is the dynamic modulus providing that the composition of the paste and type of aggregate remains the same(161).

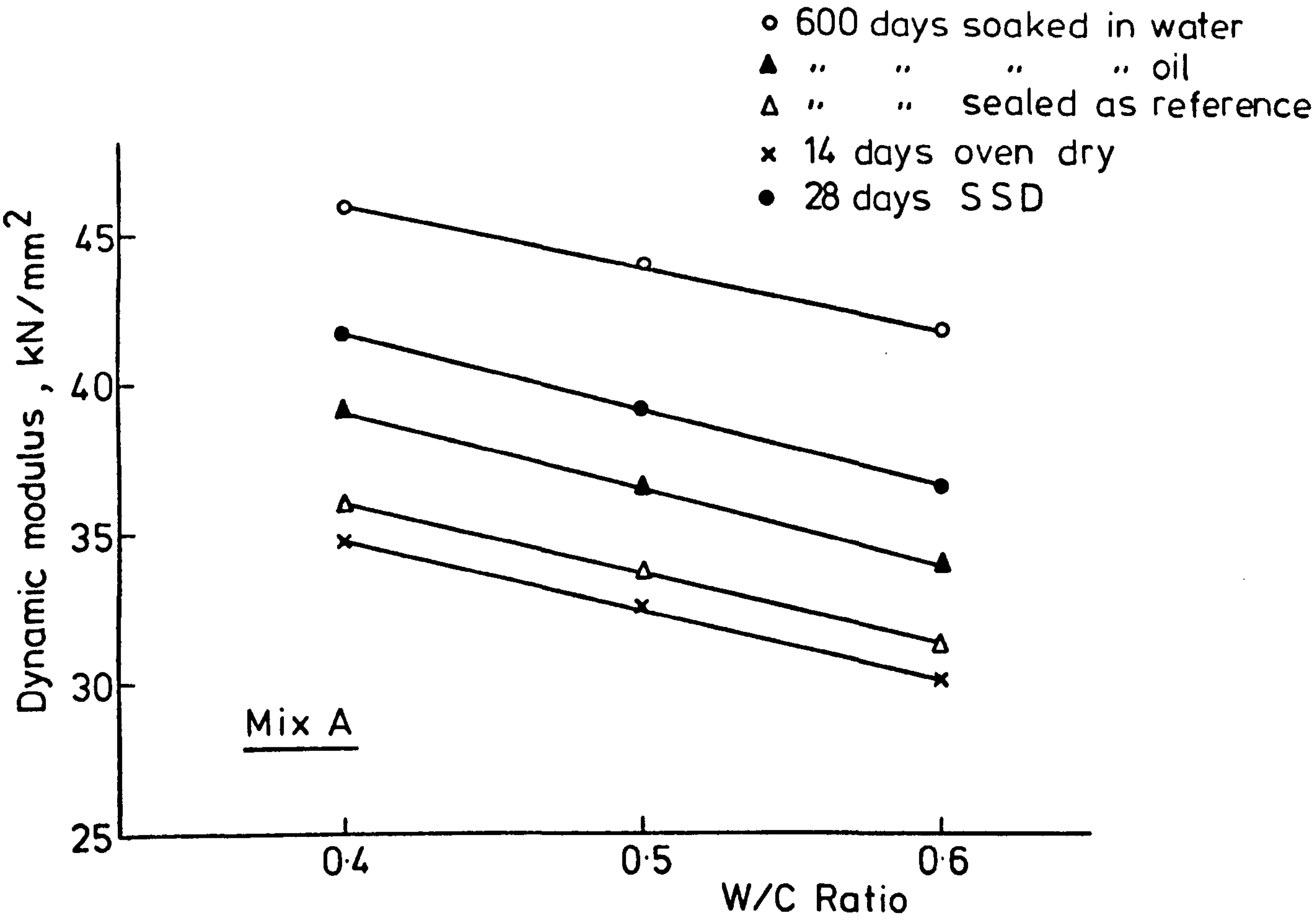


FIG. 6-13 EFFECT OF STORAGE CONDITION AND W/C RATIO ON THE DYNAMIC MODULUS OF CONCRETE , MIX A

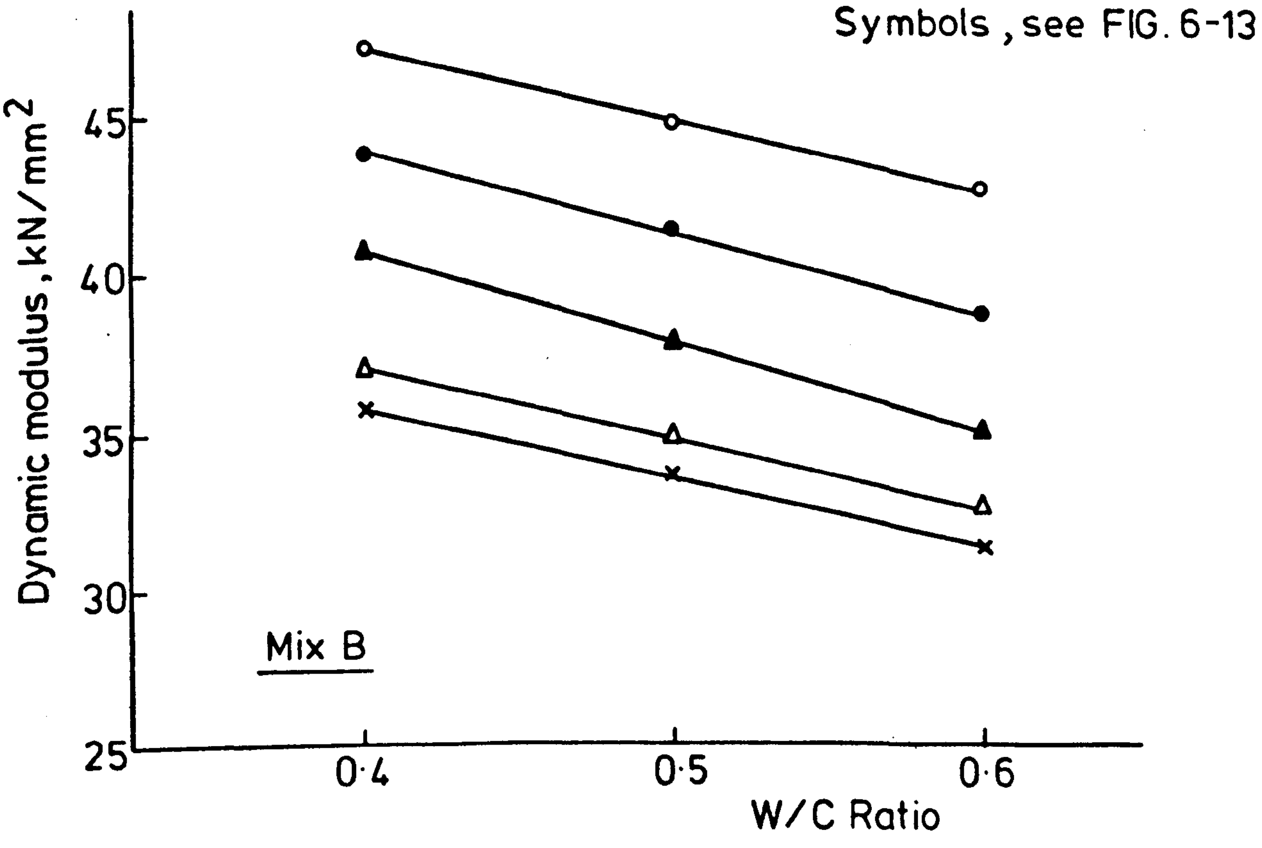


FIG. 6-14 EFFECT OF STORAGE CONDITION AND W/C RATIO ON THE DYNAMIC MODULUS OF CONCRETE , MIX B

2. The variation of the dynamic modulus with w/c ratio for the concrete stored in the three environments (crude oil, water, and sealed) is given in table (6.4). These variations show that the percentage reduction in the dynamic modulus due to the increase in water-cement ratio, was almost the same for both 10 mm and 20 mm aggregate concrete. This means that the effect of the two maximum sizes of aggregate used in this investigation was insignificant compared to the effect of the water content and the aggregate-cement ratios.

3. The relationship between the dynamic modulus (E_D) and the w/c ratio for the test results illustrated in figs. (6.13,6.14), can be expressed by the following linear equation:

$$E_D = a\ (w/c) + b \qquad \ldots\ldots \qquad (6.1)$$

where (a) and (b) are constants obtained by regression analysis and are given in table (6.5) for all concrete mixes and all types of storage conditions.

6.5.1.5 The Relation Between the Resonant Frequency and Storage Conditions.

Figs. (6.15, 6.16, 6.17) show the changes in the resonant frequency relative to 14-days oven dried specimens. These figures suggest the following:

1. The variation in frequency with age took the same general trend as that of dynamic modulus. For water soaked specimens, the increase was very rapid at the initial stages and then continues at a decreasing rate with time. The rate of increase for the sealed and oil soaked specimens, was very gradual and at a slower rate depending on the amount of absorbed oil and the consequent changes in the internal structure of the concrete specimens.

Mix Designation	W/c Ratio	% E_D of any W/c ratio to that of W/c = 0.4			
		21-days water-cured	580-days Sealed	580-days Soaked in oil	580-days Soaked in water
A	0.4	100.00	100.00	100.00	100.00
	0.5	96.71	94.10	93.59	93.56
	0.6	92.50	86.96	87.11	90.71
B	0.4	100.00	100.00	100.00	100.00
	0.5	97.10	94.10	92.78	94.75
	0.6	93.88	87.68	86.01	90.41

Table (6.4) Effect of W/c ratio on the dynamic modulus at different ages.

Mix Designation	Storage Condition	a	b	r
A	7-days water cured.	-25625	51891	99.96
	14-days oven dried.	-22980	43862	99.80
	580-days sealed.	-23465	45448	99.87
	580-days soaked in oil.	-25130	49047	99.99
	580-days soaked in water.	-21385	54638	99.98
B	7-days water cured.	-22435	52909	99.80
	14-days oven dried.	-19725	43730	99.98
	580-days sealed.	-22845	46267	99.97
	580-days soaked in oil.	-28490	52117	99.98
	580-days soaked in water.	-22600	56122	99.85

Table(6.5) Constants of regression - Equation (6.1) Dynamic modulus Vs. W/c ratio.

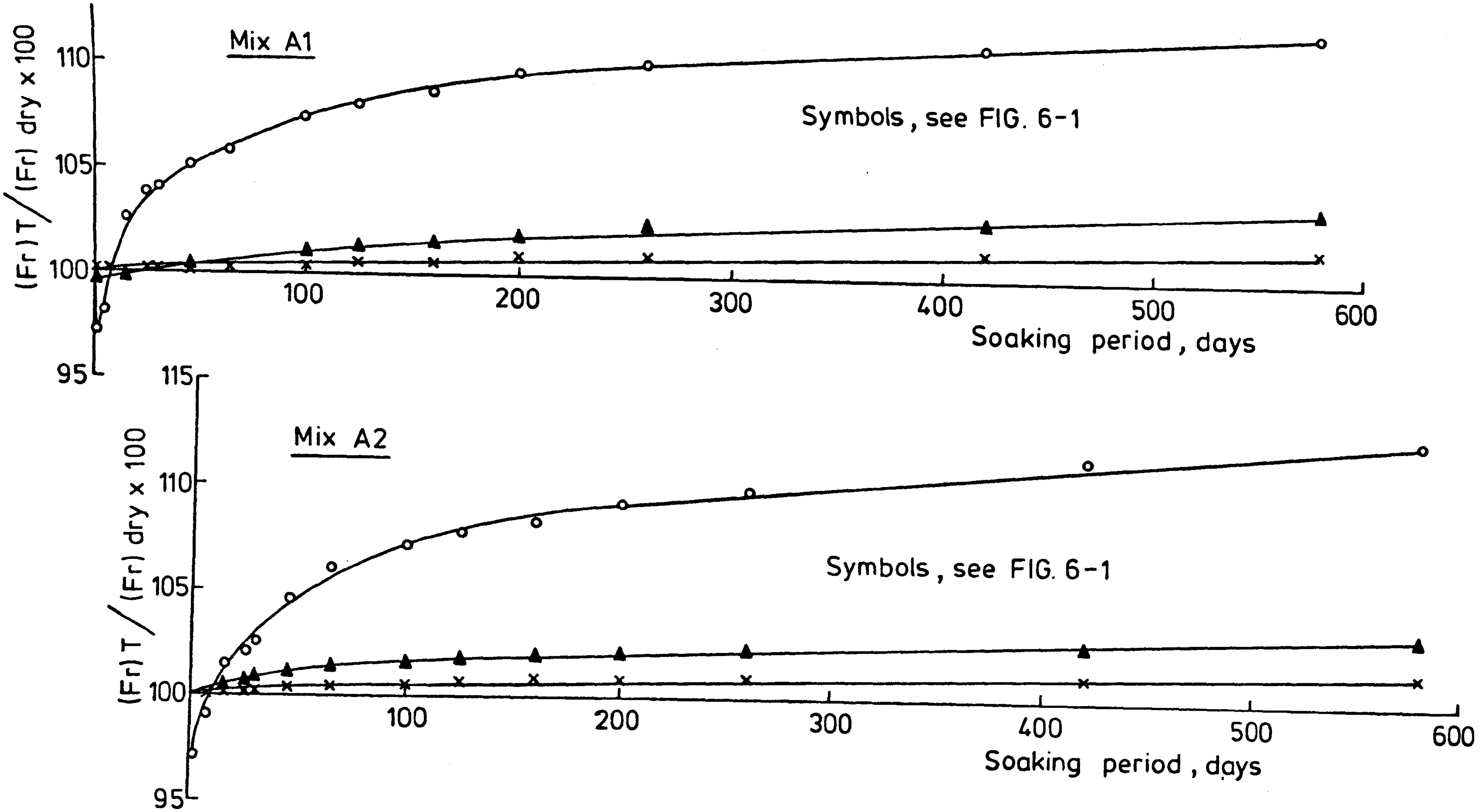


FIG. 6-15 RELATIVE CHANGE IN LONGITUDINAL RESONANT FREQUENCY (Fr) OVER THAT DRIED IN OVEN FOR 14 DAYS Vs. AGE

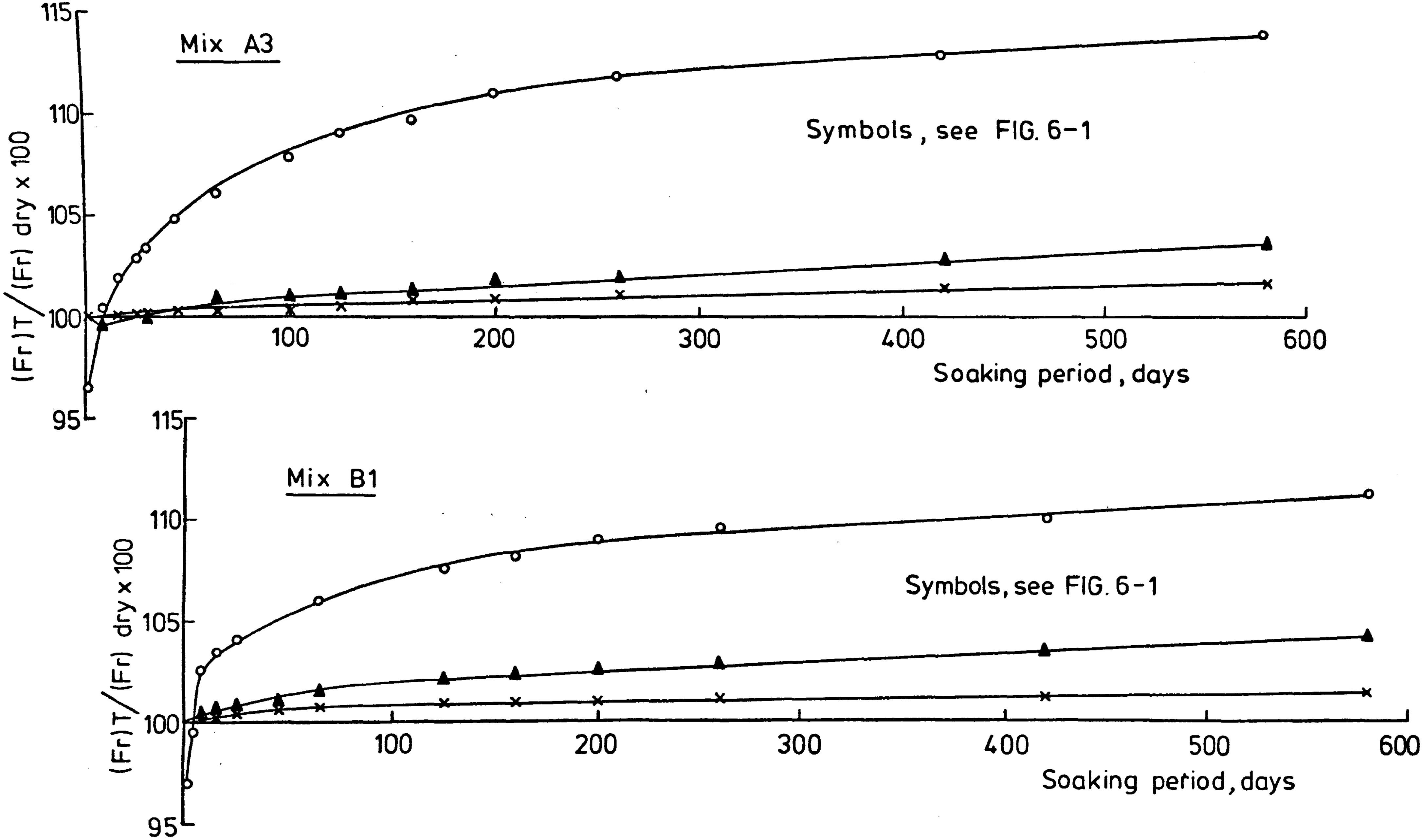


FIG. 6-16 RELATIVE CHANGE IN LONGITUDINAL RESONANT FREQUENCY (Fr) OVER THAT DRIED IN OVEN FOR 14 DAYS Vs. AGE

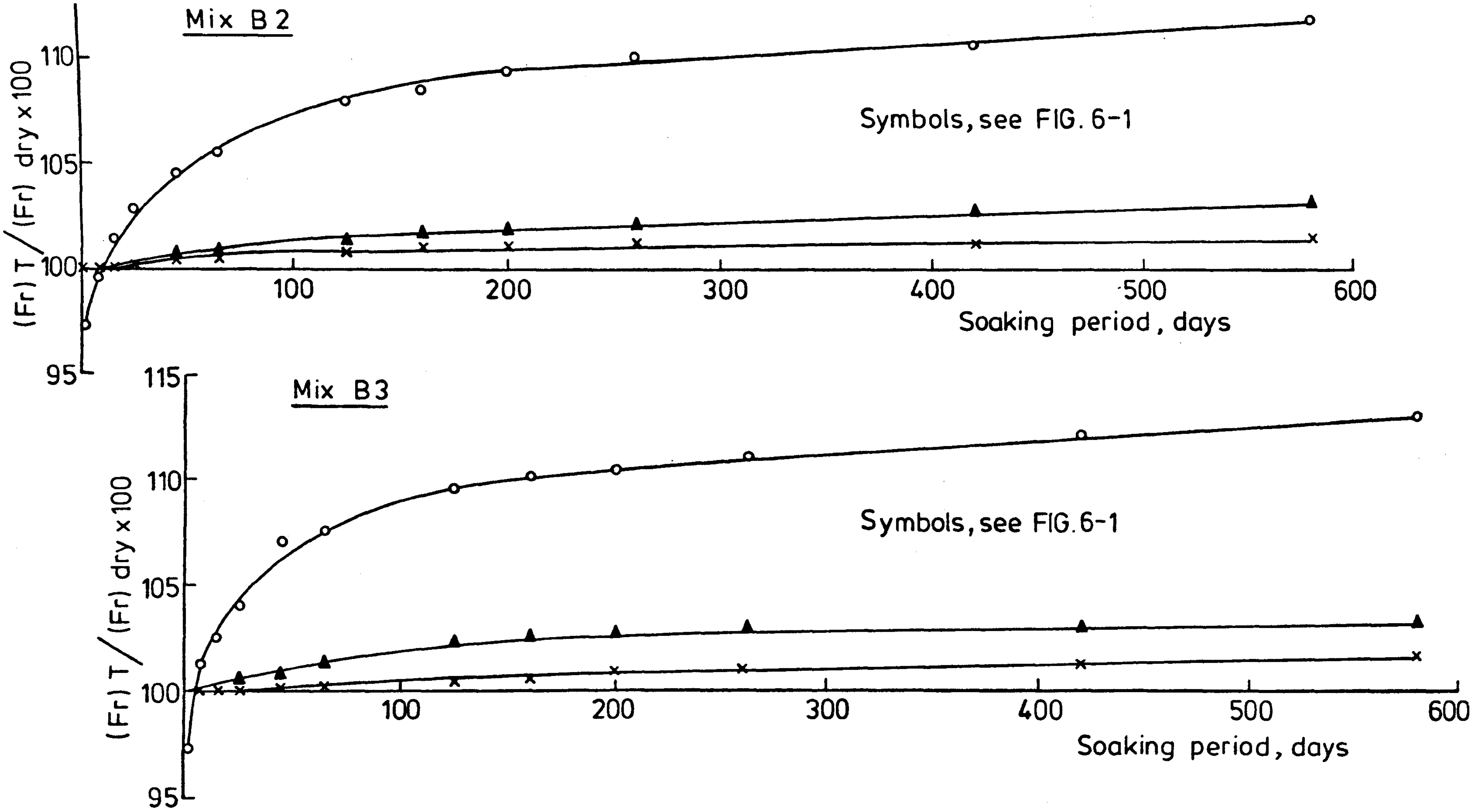


FIG. 6-17 RELATIVE CHANGE IN LONGITUDINAL RESONANT FREQUENCY (Fr) OVER THAT DRIED IN OVEN FOR 14 DAYS Vs. AGE

2. The resonant frequency values were more sensitive to moisture content variation than was the dynamic modulus. The resonant frequency of the dry concrete specimens reduced by 2.6-3.5% after soaking in water for 24 hours, which was similar to the behaviour of the compression test specimens (sec. 4.4.1). The variation in the dynamic modulus, on the other hand, was between -1.20% to +0.66% upon soaking in water for the same period, depending on the amount of absorbed water. The difference in behaviour between the resonant frequency and the dynamic modulus can be related to the method of calculating the dynamic modulus values. The equation given in section (2.7.1) for calculating the dynamic modulus, depends on the measured resonant frequency (fr) and the density (ρ). The density of the dry specimens increased by 6.4-7.2% after soaking in water for 24 hours, whilst the resonant frequency of the same specimens reduced by 2.6-3.5%, as shown in figs. (6.7) to (6.12)and figs. (6.15) to (6.17). This striking difference between the resonant frequency and the density produces the slight increase in the dynamic modulus of the dry concrete specimens after soaking in water for 24 hours.

3. The resonant frequency of the dry specimens increased by 1.3-1.7% after being sealed for 580 days, and 11-14% after being soaked in water for the same period. This increase in the resonant frequency was accompanied by an increase in the weight caused by the absorbed water which varied between 0.7-1.1% by weight for the sealed specimens and 6.4-7.2% by weight for the water soaked specimens. This variation in the amount of absorbed water affected the degree of hydration and, consequently, the resonant frequency.

4. The oven dried specimens soaked in oil for 580 days, showed an increase in the resonance frequency which varied between 3.2-4.3% (with an oil content variation of 4.54-5.50% by weight) compared to the 14 days oven dried specimens. The resonant frequency of the 580 days oil soaked specimens was between 1.70 - 2.93% higher than that of the sealed specimens, and between 6.5-9.5% lower than that of the water saturated specimens; both tested at the same age. This small increase in the resonant frequency of the oil saturated specimens may possibly be due to the same reasons explained earlier in section (6.5.1.3).

6.5.1.6 The Relationship Between the Static and Dynamic Modulus of Concrete.

It is well established(152,162,163,164) that the dynamic modulus of concrete is generally higher than the static modulus. The difference depends upon the size and shape of the specimen, age, and type of aggregate. It is also found(154,165,166) that the difference between them decreased with age.

For the present work, the relationship between the experimentally determined dynamic and static modulus is shown in fig. (6.18). The values of static modulus are those obtained from table (4.15). From figure (6.18), the following are observed:

1. There is no single relation which can satisfactorily predict the static-dynamic modulus relationship. Existing data show that such relations depend upon many factors(167), such as the water-cement ratio, aggregate-cement ratio, aggregate-cement bond, and the curing condition, and that, for more than one variable, changes in dynamic modulus do not follow the same pattern as changes in the static modulus.

2. The static modulus decreased by 6-12% with an oil content 4.51-5.72% by weight, whilst the dynamic modulus increased by about 12-14% for similar oil content. This different behaviour together with the dynamic modulus - compressive strength relationship presented in fig. (6.19), shows that the method of testing must influence the measured results.

3. The dynamic modulus for all concrete mixes and for all storage conditions is higher than the comparable static modulus. The relationship between the dynamic modulus (E_D) and the static modulus (E_S) illustrated in fig. (6.18) can be expressed by the following linear equation:

$$E_S = a\,E_D - b \qquad \ldots\ldots \qquad (6.2)$$

where a and b are constants obtained by a regression analysis and are given in table (6.6) for both concrete mixes and for all types of storage condition.

4. CP110: 1972(153) relates the dynamic modulus, E_D (kN/mm^2) to the static modulus, E_S (kN/mm^2) by the following expression:

$$E_S = 1.25\,E_D - 1.90 \qquad \ldots\ldots \qquad (6.3)$$

It is claimed that the modulus so determined falls within ± 4 kN/mm^2 of its real value when determined according to BS 1881: Part 5: 1970(103). The dynamic modulus obtained from the experimental results is compared with the predicted values, using the static modulus, table (4.15) and equation (6.3), in table (6.7). For the sealed samples, the results fall within -5.19 and + 4.89 kN/mm^2, for water soaked samples: + 0.48 and + 9.94 kN/mm^2, and for oil saturated: -2.75 and + 8.41 kN/mm^2. These rather large deviations from the expected values show that the dynamic modulus is not just a simple function of the static modulus.

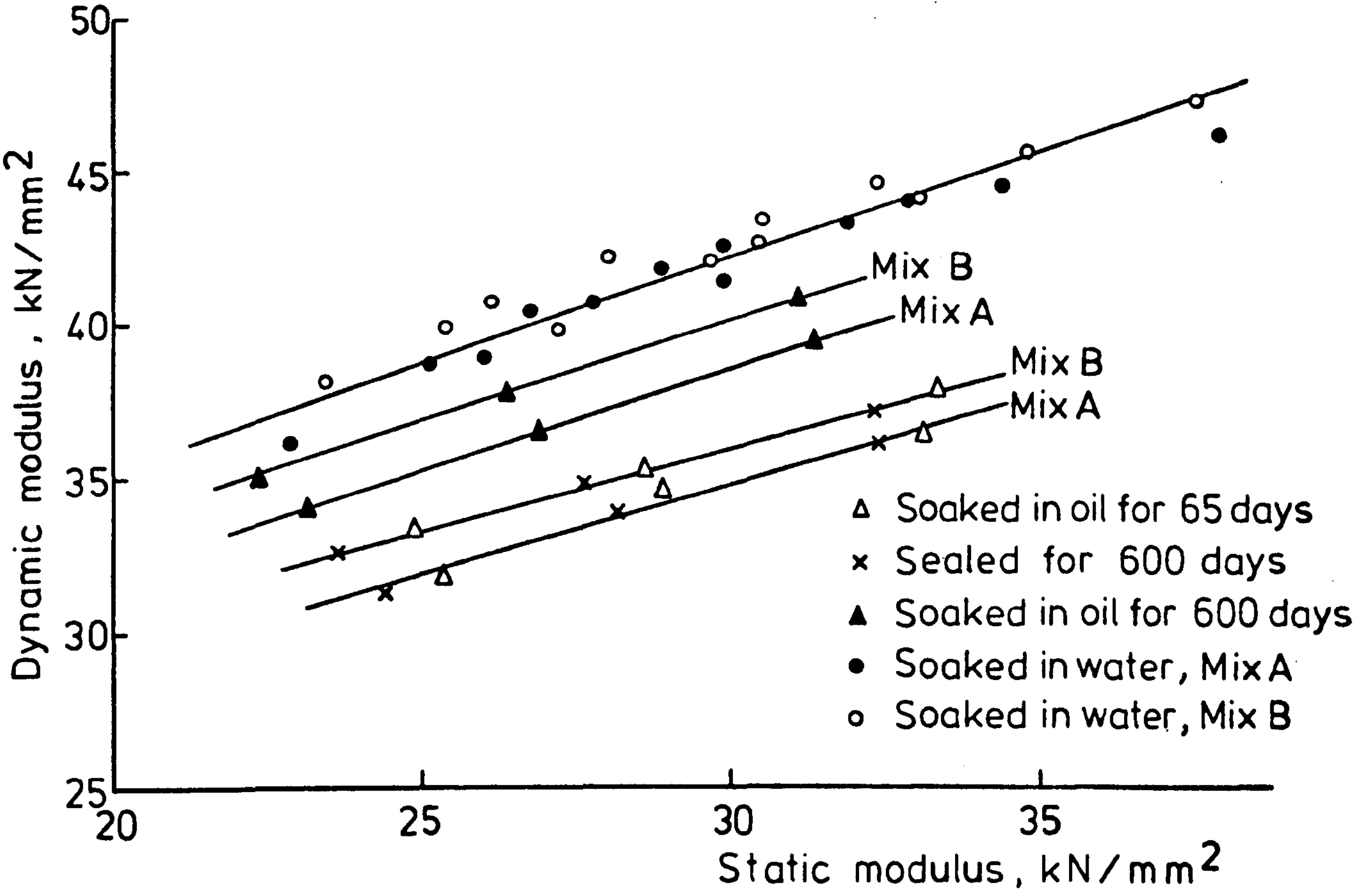


FIG. 6-18 RELATIONSHIP BETWEEN STATIC AND DYNAMIC MODULUS OF CONCRETE STORED AT DIFFERENT CONDITIONS

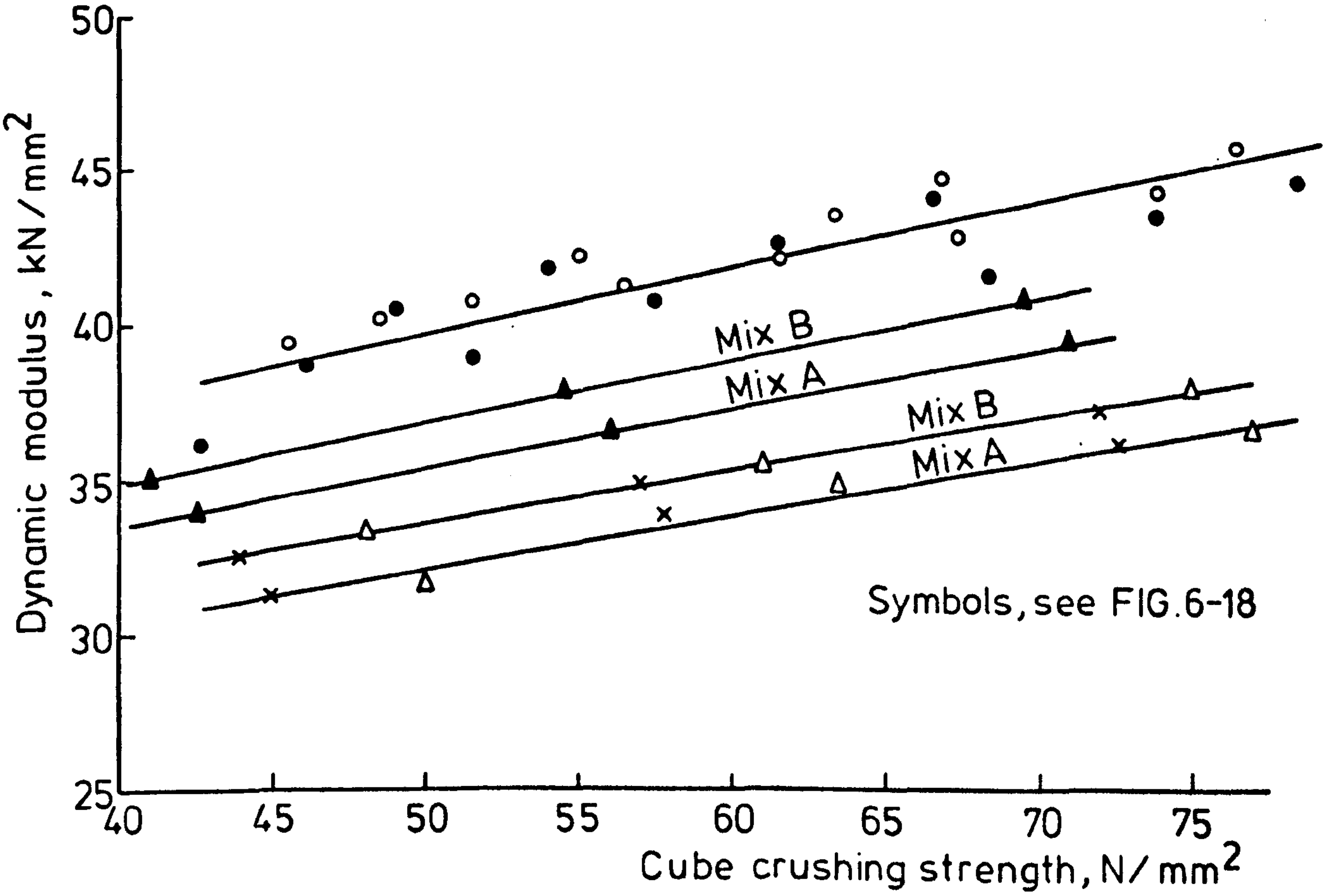


FIG. 6-19 RELATIONSHIP BETWEEN CUBE CRUSHING STRENGTH AND DYNAMIC MODULUS FOR CONCRETE STORED AT DIFFERENT CONDITIONS

70 days						160 days						580 days					
Sealed as reference		Soaked in oil		Soaked in water		Sealed as reference		Soaked in oil		Soaked in water		Sealed as reference		Soaked in oil		Soaked in water	
Code	Thesis	Code	Thesis	Code	Thesis	Code	Thesis	Code	Thesis	Code	Thesis	Code	Thesis	Code	Thesis	Code	Thesis
40.14	34.95	39.51	36.76	35.45	41.29	39.2	35.26	38.73	37.83	37.95	43.34	38.70	35.99	37.33	38.99	45.58	46.06
34.66	32.91	34.20	34.73	30.60	38.91	34.04	33.29	33.41	35.43	32.79	40.66	33.26	33.85	31.7	36.49	39.20	43.09
30.29	30.26	29.83	31.79	27.95	36.12	29.83	30.65	28.88	32.23	30.76	38.59	28.58	31.30	27.01	33.96	35.45	41.50
40.13	36.51	39.83	38.17	36.14	42.65	39.83	36.62	39.04	39.06	39.35	44.46	38.58	37.10	37.01	40.75	44.98	47.15
34.51	34.12	33.89	35.42	32.16	39.76	33.73	34.47	32.95	36.29	35.14	42.05	32.64	34.91	31.08	37.80	38.58	44.68
29.51	31.52	28.89	33.41	28.66	38.60	28.89	31.78	27.95	34.39	31.08	40.53	27.64	32.53	26.64	35.05	34.35	42.64

Table (6.7) Comparison of the experimentally obtained Dynamic Modulus with that predicted from CP110.

Mix Designation	Storage Condition	a	b	r
A	Sealed as reference	1.70	29.0	98.49
	Soaked in oil	1.64	32.7	99.84
	Soaked in water	1.36	27.8	96.60
B	Sealed as reference	1.84	36.5	99.53
	Soaked in oil	1.54	31.7	99.96
	Soaked in water	1.36	27.82	96.30

Table (6.6) Constants of regression - equation (6.2) Static Modulus Vs. Dynamic Modulus.

6.5.2 The Damping Capacity of Concrete.

The damping capacity measured as the logarithmic decrement (δ) was calculated from equation (2.8) given in section (2.7.2) The calculated values for the 7 days water cured specimens and the 14-days oven dried specimens are given in table (6.8) as the mean of three prisms with the corresponding standard deviation.

The present work studies the effect of absorbed oil on the damping capacity of oven dried specimens compared to companion specimens soaked in water or sealed as a control specimen. The damping capacity is also related to other mechanical properties of concrete.

Mix Designation	w/c ratio	7-day water curing		14-day oven drying		% Reduction due to drying
		$\delta x10^{-2}$	S.D.	$\delta x10^{-2}$	S.D.	
A1	0.4	5.79	0.11	1.18	0.18	79.6
A2	0.5	6.39	0.20	1.45	0.23	77.2
A3	0.6	7.12	0.21	1.87	0.21	73.7
B1	0.4	5.71	0.13	1.08	0.13	81.1
B2	0.5	6.14	0.18	1.21	0.17	80.3
B3	0.6	6.68	0.16	1.39	0.20	79.2

Table (6.8) The logarithmic decrement ($\delta x10^{-2}$) at the age of 7-days and after being oven dried.

6.5.2.1 Effect of W/C Ratio and the Maximum Size of Aggregate on the Damping Capacity of Concrete.

Three water/cement ratios were considered for each maximum size of aggregate and their effect on damping can be seen from tables (6.8, 6.9). The results were in agreement with the previous findings(101,106,168), i.e.,an increase in the water-cement ratio gives an increase in the damping capacity of concrete. The damping capacity of the 7-day water cured concrete was reduced by 14.52-18.68% when the water-cement ratio was reduced from 0.6 to 0.4. The damping capacity was reduced by 74-81% after drying the same specimens for 14 days in an oven at 105° C to a constant weight. The highest reduction was for the concrete of high water-cement ratio which has a higher percentage of pores.

The fact that logarithmic decrement increases with water content may show that the damping capacity of concrete is related to the porosity. The pore volume varies inversely with the water-cement ratio and the aggregate content, and, at a given age, with the degree of hydration. The change in damping capacity is greatest during the early ages and decreases as hydration proceeds.Therefore,in the early stages of hydration, the logarithmic decrements depend on water-cement ratio for concrete of all mix proportions,but as hydration and maturity increases, the logarithmic decrement becomes less dependent on water-cement ratio and the influence of water content becomes less significant.

Results presented in table (6.9) show that, the 7-days water cured logarithmic decrement (δ) of the 20 mm aggregate concrete was 1.5-6.0% less than that of the 10 mm aggregate concrete cured for the same period. The reduction was

Mix Designation	w/c Ratio	Storage Condition	Logarithmic Decrement ($8x10^{-2}$) at the following ages													
			7-day Moist-Curing	14-day Oven Drying	Soaking period after oven drying (days)											
					1	3	7	15	25	65	125	165	200	265	420	580
A1	0.4	Sealed	5.79	1.18	1.40	1.44	1.45	1.52	1.60	1.85	2.15	1.95	1.85	1.90	1.98	2.02
		Soaked in oil	5.79	1.18	1.74	1.75	2.25	3.05	3.70	5.16	7.07	8.36	7.65	7.85	7.70	7.39
		Soaked in water	5.79	1.18	5.15	6.54	6.65	6.90	7.15	8.05	7.85	6.86	6.34	5.88	5.55	5.45
A2	0.5	Sealed	6.39	1.45	1.44	1.48	1.54	1.55	1.51	1.66	1.90	1.92	1.98	2.20	2.05	2.10
		Soaked in oil	6.39	1.45	1.73	2.00	2.60	3.55	3.88	5.36	6.99	8.79	8.15	7.99	7.95	7.50
		Soaked in water	6.39	1.45	5.55	6185	7.15	7.25	7.49	8.35	7.35	6.90	6.80	6.45	6.05	5.95
A3	0.6	Sealed	7.12	1.87	2.04	1.88	1.90	1.96	2.00	2.15	2.25	2.48	2.40	2.45	2.30	2.25
		Soaked in oil	7.12	1.87	1.88	2.14	2.39	2.90	3.27	5.73	7.25	9.51	8.85	8.55	8.60	8.15
		Soaked in water	7.12	1.87	6.86	6.91	7.55	8.10	8.28	8.85	7.95	7.85	7.70	7.45	7.15	6.85
B1	0.4	Sealed	5.72	1.08	1.28	1.23	1.25	1.48	1.51	1.62	1.75	1.80	1.91	1.98	1.92	1.88
		Soaked in oil	5.72	1.08	1.79	2.21	2.30	3.29	3.50	4.68	7.25	8.25	7.55	7.68	7.52	7.08
		Soaked in water	5.72	1.08	5.45	6.25	7.05	7.25	1.50	7.75	7.05	7.02	6.41	5.38	5.33	5.23
B2	0.5	Sealed	6.14	1.21	1.37	1.30	1.33	1.40	1.57	1.60	1.75	1.65	1.75	1.92	1.90	1.95
		Soaked in oil	6.14	1.21	1.99	2.42	2.84	4.50	5.05	7.01	7.90	8.55	7.70	7.85	7.75	7.45
		Soaked in water	6.14	1.21	5.60	6.25	6.80	7.21	7.85	8.15	7.45	7.30	7.15	6.45	6.05	5.80
B3	0.6	Sealed	6.64	1.39	1.36	1.44	1.45	1.60	1.60	1.75	2.05	2.10	2.00	1.95	1.92	1.93
		Soaked in oil	6.64	1.39	2.46	2.55	3.17	4.14	4.75	7.07	8.66	9.20	8.85	8.16	8.35	7.95
		Soaked in water	6.64	1.39	6.11	6.15	6.55	7.15	7.95	8.55	7.85	7.65	7.55	7.05	6.95	6.55

Table (6.9) Changes of logarithmic decrement ($8x10^{-2}$) with age for concrete specimens soaked in crude oil, water, and sealed as reference.

significant ($P \leqslant 0.05$) for the concrete of 0.6 w/c ratio only. Thus it appears that aggregates with a large specific area produce greater damping. Previous results(168) have shown that small-sized aggregates would have a higher damping than large-sized aggregates. The aggregate may affect the damping by reducing the matrix volume content and the resulting percentage of pores in the concrete(161). The bond between the aggregate and the matrix can also influence the damping characteristics of concrete, and a weaker bond may produce a concrete with increased damping capacity(108). Many(34,42,108,147) investigators have found that increasing the maximum size of the aggregate decreases the bond strength which agrees with the results of the present investigation.

6.5.2.2 The Effect of Age on the Damping Capacity of Concrete.

Figs.(6.20) to (6.25) show the variation in the logarithmic decrement with time for the oven dried specimens soaked in crude oil, water, or sealed as a reference. From these results the following observations can be made:-

1. The damping capacity for these concrete specimens soaked in water and those in crude oil, increased very rapidly at the initial stages of soaking due to the high initial rate of penetration of absorbed liquid, but then decreased with age, as explained later in section (6.5.2.3).
2. At the early stages of the soaking period, the rate of increase in the damping capacity of water soaked specimens was higher than that of the oil soaked specimens because of the higher rate of water penetration compared to crude oil. After 24 hours of soaking, the damping capacity of the water soaked

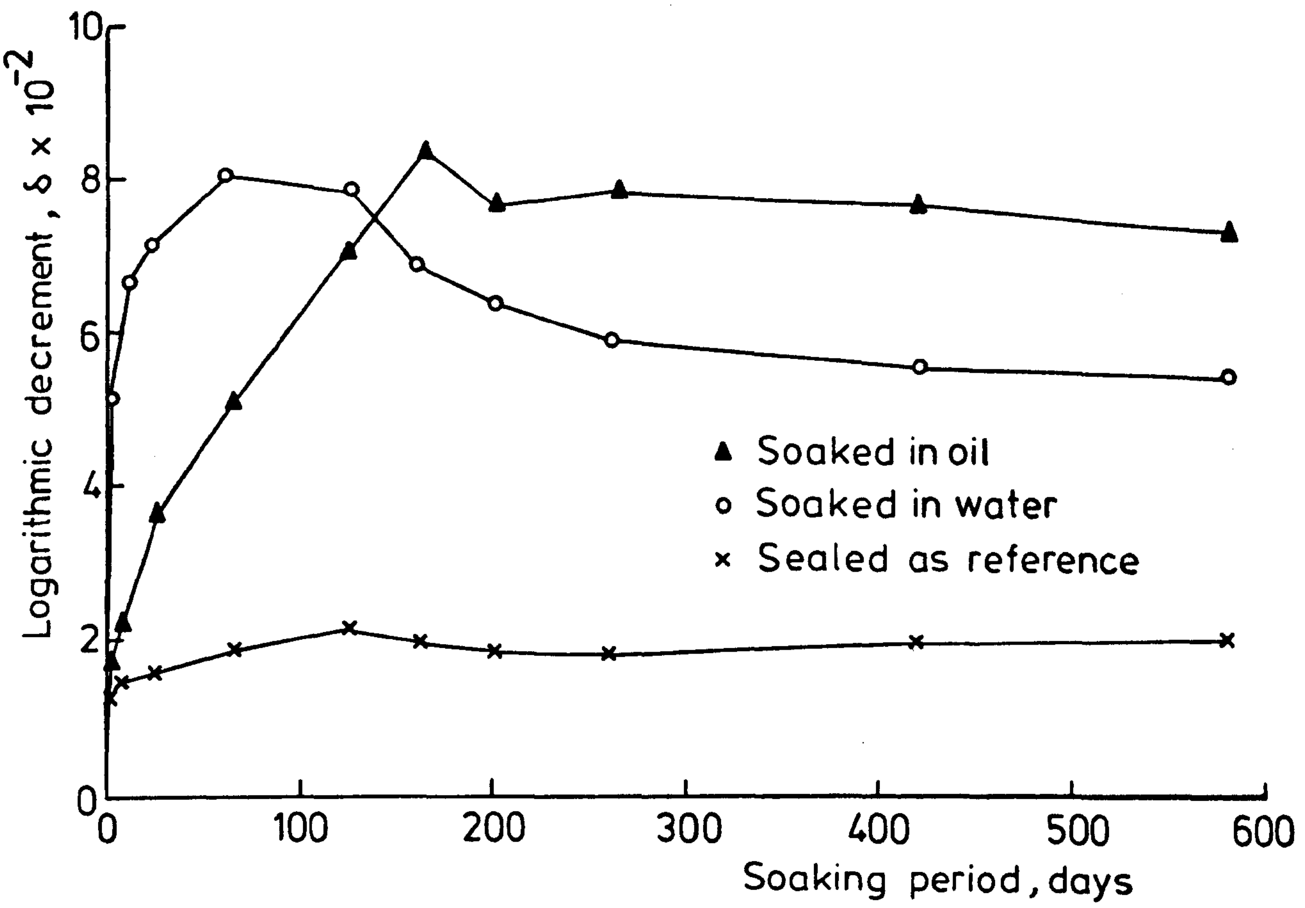


FIG. 6-20 LOGARITHMIC DECREMENT, δ Vs. AGE, MIX A1

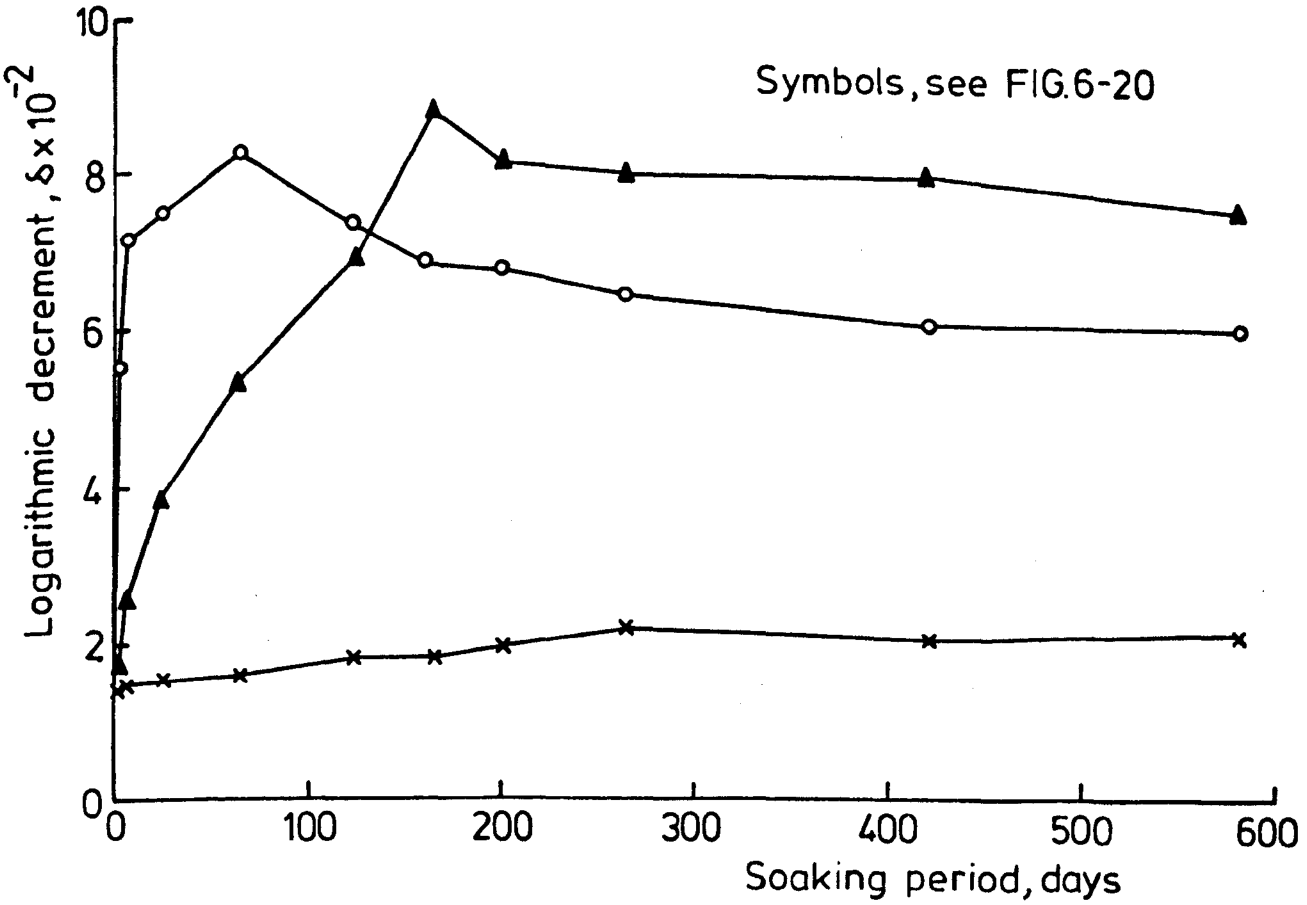


FIG. 6-21 LOGARITHMIC DECREMENT, δ Vs. AGE, MIX A2

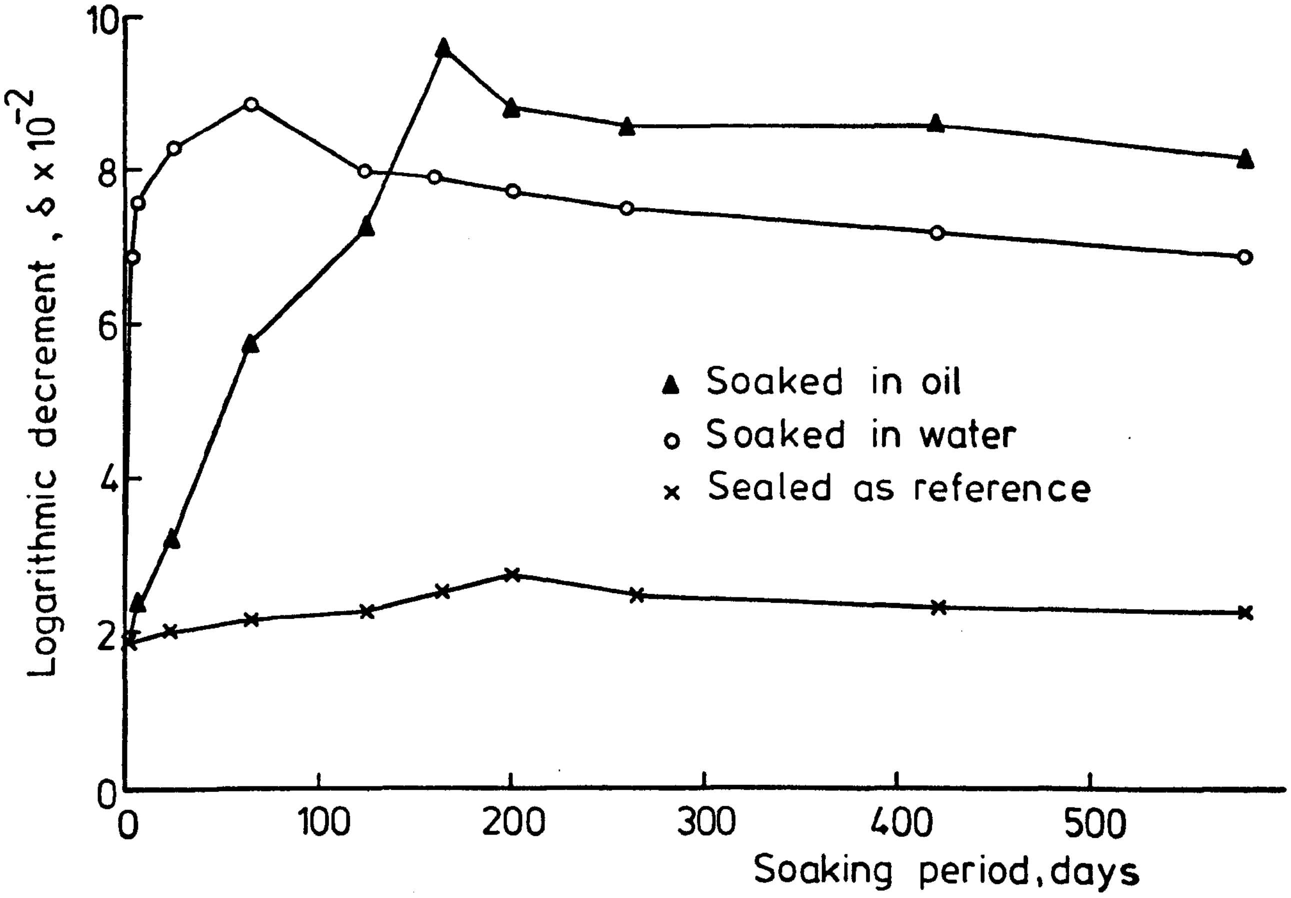


FIG. 6-22 LOGARITHMIC DECREMENT, δ Vs. AGE, MIX A3

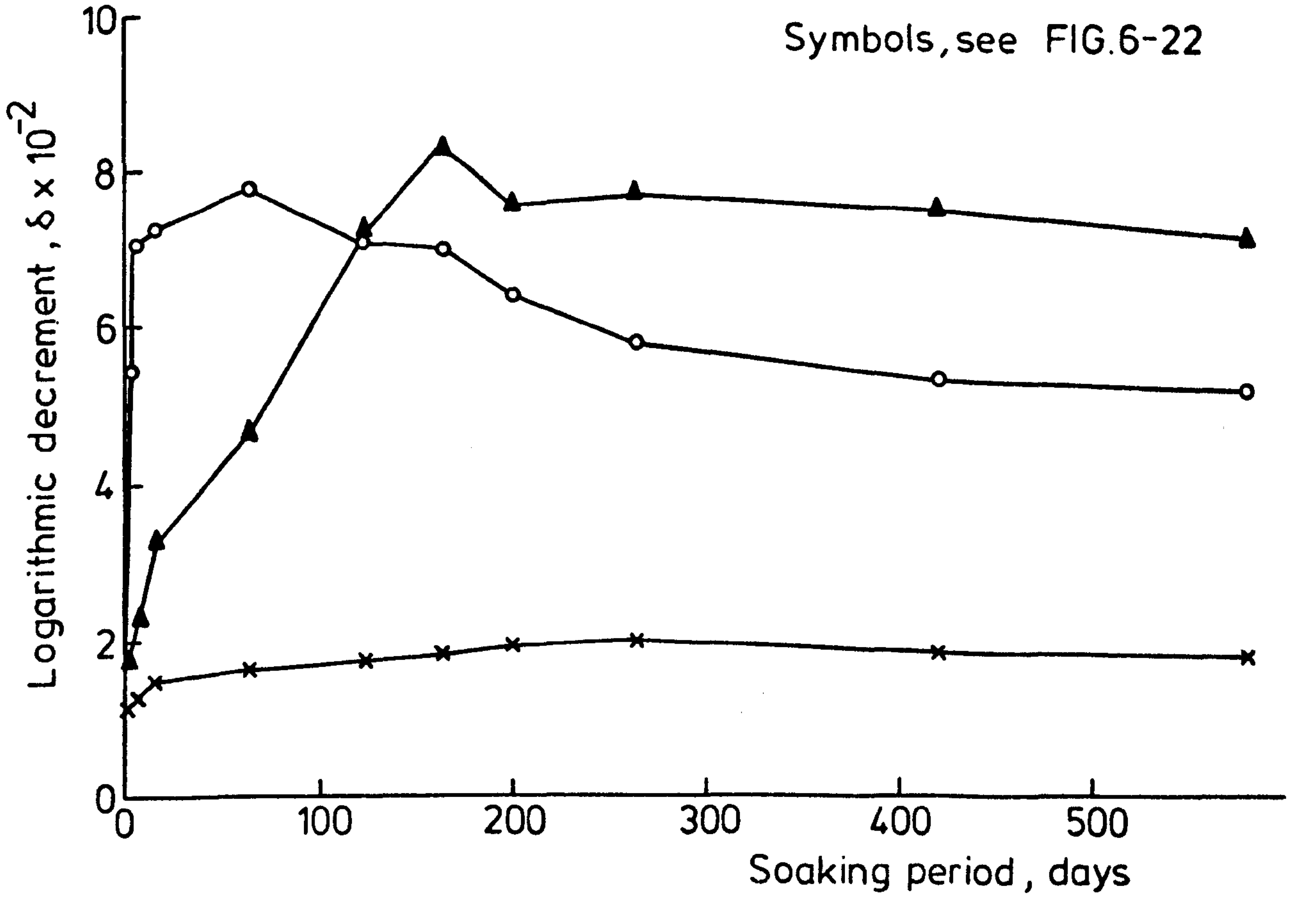


FIG. 6-23 LOGARITHMIC DECREMENT, δ Vs. AGE, MIX B1

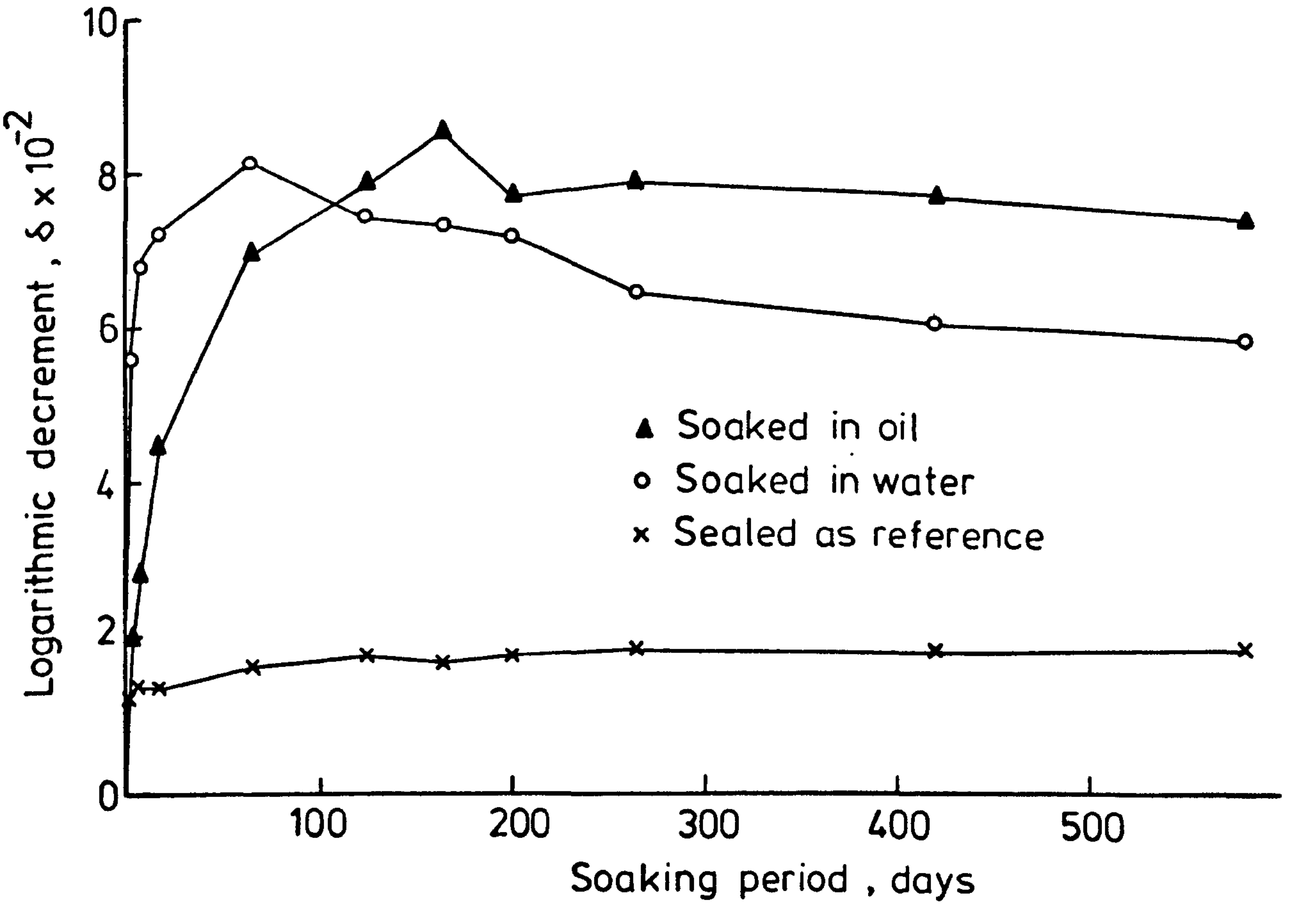


FIG. 6-24 LOGARITHMIC DECREMENT, δ Vs. AGE (MIX B2)

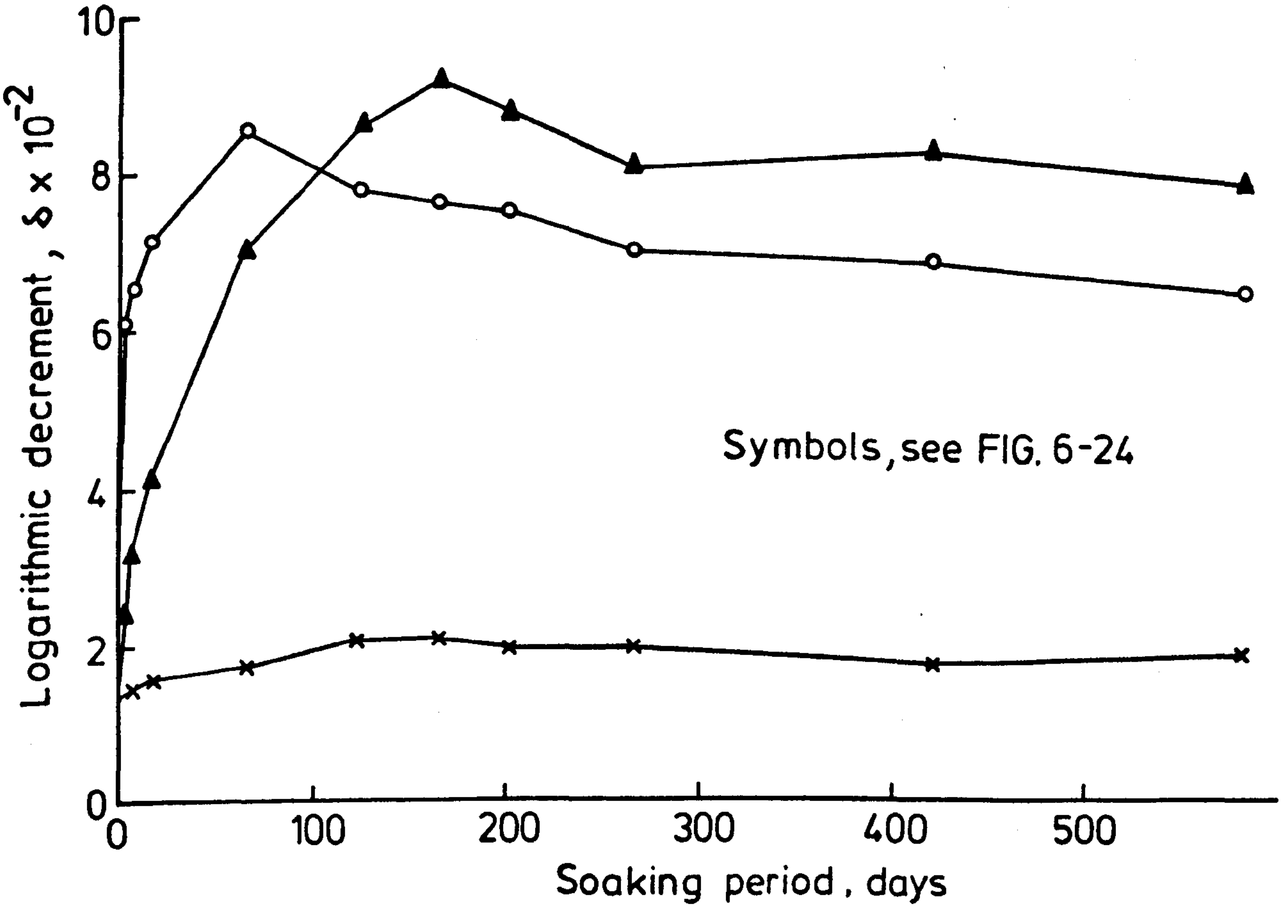


FIG. 6-25 LOGARITHMIC DECREMENT, δ Vs. AGE (MIX B3)

specimens increased by 5.15-6.86% compared to an increase of only 1.74-2.46% for the crude oil soaked specimens. This high increase in the damping was accompanied by high water absorption ranging between 5.07-6.31% by weight, whilst the oil soaked specimens only absorbed 0.50-0.70% of oil by weight.

3. The damping capacity reached its maximum value at about 65 and 160 days for the water soaked specimens and the oil soaked specimens respectively, after which it started to decrease with the increase in the soaking period. The reduction was between 23-35% for the water saturated specimens and 11.5-14.5%for the oil saturated specimens, after a soaking period of 580 days compared to the maximum damping capacity.

4. The test results presented in table (6.9) showed that the reduction in the damping capacity was higher for concrete of higher cement content (w/c = 0.4). This might be due to the fact that the decrease in the damping capacity was a result of the resumption of hydration and autogenious healing of some of the cracks, and thus, the concrete with high cement content and w/c > 0.38 (28) would develop a high degree of hydration, lower porosity, and lower damping capacity.

5. The 14-day oven dried specimens showed a relatively large increase in the damping capacity which varied between 54-84% after being stored in the open air in the laboratory for about 165-265 days,depending on the amount of absorbed water from the surrounding environment. The amount of water absorbed was between 0.58-0.74% by weight, the highest amount of absorbed water was for the concrete mix of the highest porosity. This

shows that damping is more sensitive to small moisture change in a dry specimen than in a water cured specimen. This increase in damping capacity was then followed by a gradual decrease, which varied between 5.50-5.75% after a storage period of 580 days compared to the maximum damping capacity.

6.5.2.3 The Effect of Water and Oil Penetration on the Damping Capacity of Dry Concrete.

The test results presented in the previous section showed that the damping capacity for all specimens increased rapidly at the initial stages, reaching a maximum value and then started to decrease gradually depending on the storage condition and the amount of absorbed liquid, oil or water. The increase in the damping capacity of dry specimens sealed for 65 days was between 10-25 times less than similar specimens soaked in water. These variations in the results suggest that the moisture content is the most important factor influencing damping.

The variation in the damping capacity of dry concrete samples soaked in water can, therefore, be divided into two stages; the first stage exhibiting very rapid increase in the damping capacity accompanied by a high rate of water penetration refilling the emptied pores in the matrix. In the second stage, the rate of penetration reduced significantly and the soaked specimens become almost fully saturated with absorbed water, so that as hydration and maturity proceeded the logarithmic decrement became less dependent on the water content and the damping decreases.

The dry specimens soaked in oil follow the same general trend as the water soaked specimens with two major differences, as follows:

1. The damping capacity took longer to reach its maximum value due to the slower rate of oil penetration.

2. After a soaking period of 580 days, the reduction in the damping capacity of oil soaked specimens as a percentage of its maximum value was between 38-64% less than that for the water soaked specimens, which might be for the following reasons:

a. It appears(168) that most of the damping in concrete occurs in the matrix, and at some of the interfacial boundaries, and less in the aggregate. The presence of crude oil is found to reduce the aggregate-cement bond as discussed earlier in chapter 4, section (4.5.2), and a weaker bond is found to give a higher damping capacity(34).

b. The specimens soaked in oil cannot continue to hydrate or cracks heal.

6.5.2.4 The Relation Between Logarithmic Decrement, Compressive Strength and Dynamic Modulus of Concrete.

The variation of logarithmic decrement of oil and water soaked concrete specimens with the cube crushing strength is shown in fig. (6.26) for two maximum sizes of aggregate (10 and 20 mm), three water-cement ratios, and three different storage conditions. The figure shows a decrease in the damping capacity with increasing cube crushing strength, but the scatter is too great to define any precise relation. It is also shown that there is no single relationship between the damping capacity and the cube crushing strength for concretes from different storage conditions, moisture content, and maximum size of aggregate. This indicates that the damping

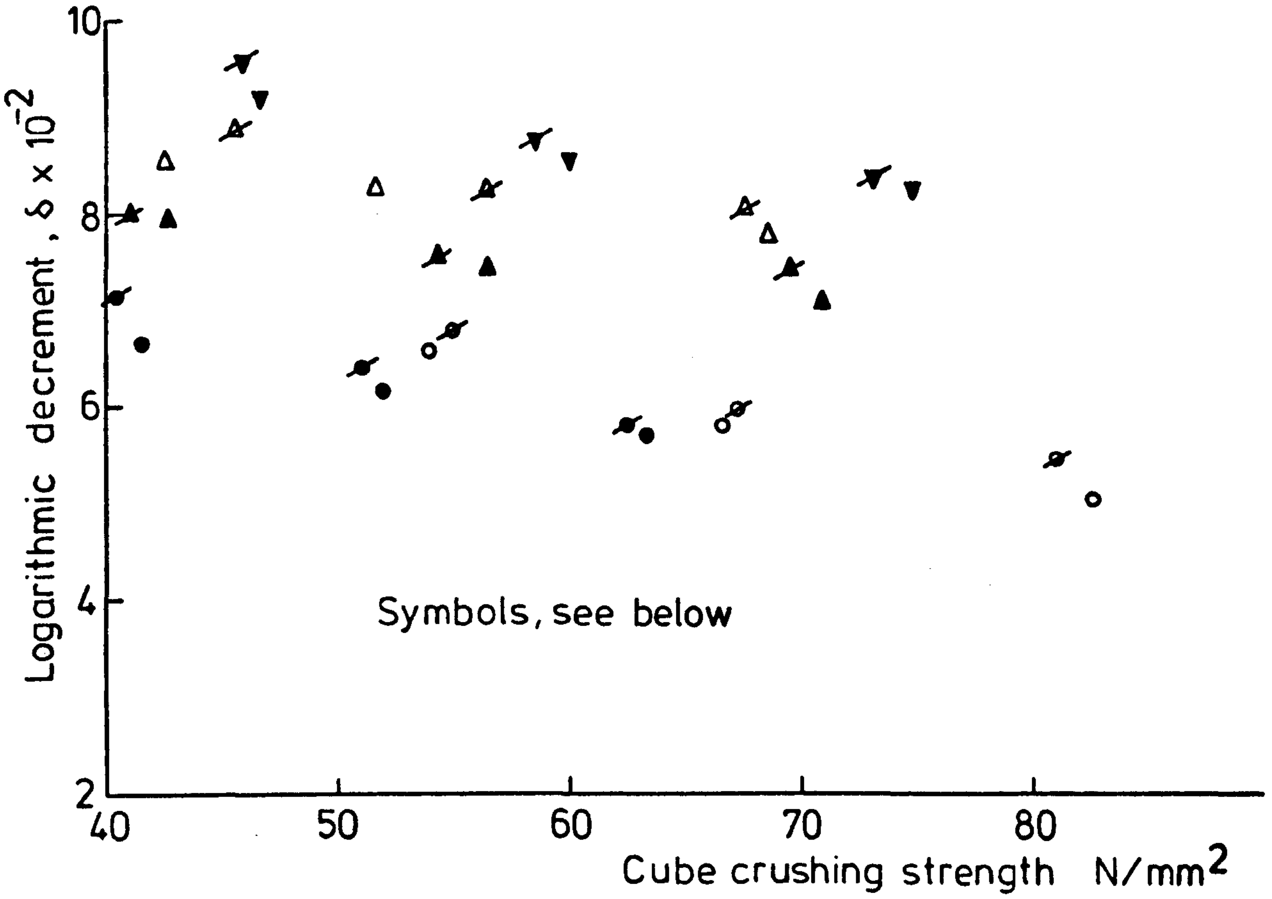


FIG. 6-26 THE RELATIONSHIP BETWEEN LOGARITHMIC DECREMENT, δ AND CUBE CRUSHING STRENGTH FOR WATER AND OIL SOAKED SPECIMENS

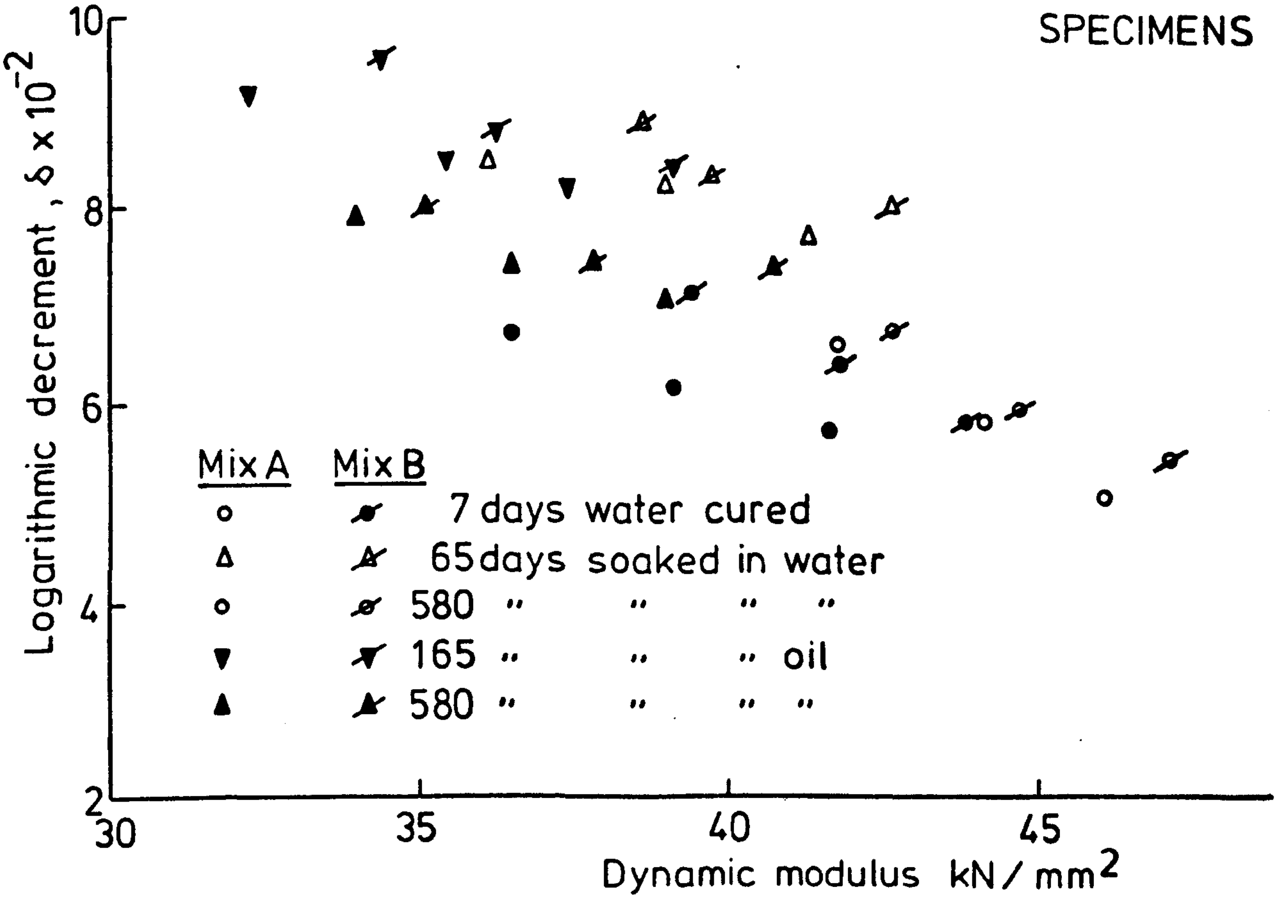


FIG. 6-27 THE RELATIONSHIP BETWEEN LOGARITHMIC DECREMENT, δ AND DYNAMIC MODULUS FOR WATER AND OIL SOAKED SPECIMENS

capacity is more dependent on the moisture content of the concrete than is compressive strength.

The variation of damping with dynamic modulus of oil and water soaked concrete is shown in fig. (6.27). The results show that the damping capacity decreases with an increase in dynamic modulus, which is expected since damping also decreases with an increase in compressive strength, but there is a considerable scatter of results and no unique relationship can be obtained.

CHAPTER 7.

CONCLUSIONS, LIMITATIONS OF THE PRESENT WORK, AND PROPOSALS FOR FUTURE WORK.

7.1 CONCLUSIONS.

7.1.1 Mechanical properties.

7.1.1.1 HCP and Mortar.

1. The strength of dry cube specimens was reduced by between 40-60% for HCPs and 35-45% for mortars after 24 hours of soaking in water. This reduction was then followed by an initially high rate of strength gain which gradually decreased with time. The oil saturated specimens did not show any significant reduction after being soaked in oil for 24 hours.

2. The cube crushing strength of oil soaked specimens decreased with an increasing amount of absorbed oil. The maximum reduction was between 4-8.5% for HCPs and 4.5-14% for Mortars after a soaking period of 300 days. Specimens of lower porosity had a smaller percentage strength reduction.

3. The compressive strength and elastic modulus of HCPs and Mortar can be related exponentially to w/c ratio.

4. The elastic modulus of the HCP specimens initially dried and then resaturated in water for 300 days and that of the companion sealed specimens, were higher than the modulus of oil saturated specimens by about 20-26% and 7-10% respectively. The elastic modulus of initially dry

specimens later soaked in oil for 300 days was between 11.5-15% for 1:2 Mortar and 11.5-16.5% for 1:3 Mortar less than that of the corresponding sealed specimens.

7.1.1.2 Plain Concrete.

1. The strength of oven dry concrete specimens was reduced by 25-36% for cubes, 29-37% for cylinders, and 26-38% for prisms after 24 hours of soaking in water. This sharp reduction was then followed by an initially high rate of strength gain which gradually decreased with time, but the strength never reached that of the continuously wet cured concrete.
2. There was no immediate reduction of strength when the dry specimens were soaked in crude oil. The reduction in the strength increased as the specimens were kept continuously soaked in oil for longer periods due to the increasing amount of absorbed oil. After a soaking period of 600 days, the reduction varied between 9.5-22.5% for cubes, 10-19.5% for cylinders, and 8-18% for prisms. The percentage reduction was higher in the specimens of higher total porosity due to the high amount of absorbed oil.
3. The average rate of strength reduction due to storage in crude oil decreased with time. The average rate of annual reduction was between 5.78-13.69% for cubes, 6.08-11.86% for cylinders, and 4.87-10.95% for prisms.
4. The compressive strength of cubes soaked in crude oil for 600 days was between 1.5-6.5% and 13-25% lower than the strength of the companion cubes sealed as a reference and soaked in water respectively. This reduction was only

significant ($P \leq 0.05$) when compared with the water saturated specimens. For the cylinders and prisms, the percentage reduction in the compressive strength due to soaking in oil for 600 days was between 16-28% lower than the companion water soaked specimens and was almost the same as that for the companion sealed specimens.

5. The tensile strength of the oven dried specimens soaked in oil showed a continuous reduction with time. The highest reduction after a soaking period of 750 days was between 6.5-9.5% and was almost the same for all types of tensile tests used in this investigation.
6. The tensile strength of oil saturated specimens was between 5.5-9.5% and 17-21.5% lower than that of water saturated specimens for the cylinder splitting tests and flexural tests respectively. The percentage reduction due to absorption of oil compared to that of the companion sealed specimens was almost the same for concrete mixes of low w/c ratio but reduced as the w/c ratio increased, i.e. porosity increased and the amount of absorbed oil increased.
7. At low loads, the behaviour of the oil saturated and the companion control prisms was similar, but at higher loads crude oil saturated concrete became weaker but more flexible with a higher Poisson's ratio, volumetric expansion and longitudinal strain.
8. Crude oil saturation decreased the elastic modulus. This reduction varied between 6-11% within a soaking period of 600 days compared with that of the dry concrete before soaking in oil. This reduction was significant for the 20 mm maximum size aggregate concrete independent of w/c ratio

but only for the 10 mm maximum size aggregate concrete using w/c = 0.6.

9. The dried concrete specimens soaked in crude oil shrank slightly during a soaking period of 25 days and then began to swell with time at a very slow rate but the initial shrinkage was not recovered after a soaking period of 560 days. Similar companion specimens kept in water or stored in the laboratory continued to swell with time. The amount of swelling depended on the amount of absorbed water.

7.1.2 Oil Absorption.

1. For hardened cement paste and concrete, the amount of absorbed water was higher than the amount of water lost on drying to 105° C, partly because extra cracks may have been created during the drying and cooling processes.
2. The total volume of absorbed oil was less than the volume of water lost during the 14 days oven drying by about 18-28% for HCPs, 6-18% for mortars, and 0.5-6.4% for concrete. This means that full saturation was never achieved and some voids left on drying were unfilled by the oil.
3. Most oil absorption occurred within a storage period of 4 months for HCPs and mortars and six months for concrete. The mix with a higher w/c ratio had a higher porosity and therefore had the highest absorption.
4. The depth of penetration and area of concrete saturated with crude oil were highly dependent on the porosity and can be exponentially related to the duration of the test.

7.1.3 Bond Strength.

7.1.3.1 Pull-Out Test.

1. The average bond strength of oil soaked specimens decreased with the soaking period. This reduction was between 15.5-21% for the mild round steel bar and 8.5-9.5% for the deformed bar after a soaking period of 750 days compared to that of 120-days oil soaked bond strength.
2. The rate of reduction of the average bond strength due to storage in crude oil was small and reduced with time. The average rates of annual reduction were 8.98-12.17% for the plain bar and 4.92-5.50% for the deformed bar.
3. The average bond strength of the 750 days oil soaked plain bar specimens was between 13.5-26.5% and 43-51% lower than the companion sealed and water soaked specimens respectively. For the deformed bars, the reduction was between 13.5-19.5% and 34-36% for the sealed and water soaked specimens respectively.
4. The critical bond stress corresponding to a free end slip of 0.05 mm was lowest for the oil saturated specimens depending on the type of the reinforcing bar and the amount of absorbed oil.
5. The average bond strength of the dry specimens soaked in water for 750 days showed a gradual increase with time. The average rates of annual increase were between 20.68-21.41% for the plain bar specimens and 10.10-12.53% for the deformed bar specimens.
6. The average bond strength of the deformed bar was about 4 times greater than that of the plain round steel bar.

7.1.3.2 Modified Pull-Out Test.

1. For the same slip, the oil saturated specimens developed lower bond strength than the sealed specimens. The dry specimens soaked in water developed the highest bond strength.
2. Both the sealed and oil saturated specimens had more loaded end slip before failure compared to that of water saturated specimens tested at the same age.
3. The critical bond stress of the oil saturated specimens corresponding to loaded end slip of 0.025 mm was 35-41% and 72% lower than that for the sealed and water soaked specimens respectively.
4. The bond stress distribution along the plain bar was not uniform for all the curing conditions used in this investigation. The maximum local bond stress occurred near the loaded end of the plain bar at relatively low loads and moved towards the embedded end as the applied load was increased.
5. For the specimens initially dried and then soaked in water, the bond stress at both ends of the bar tended to become equal as the applied load approached the maximum value. For the sealed and oil saturated specimens the bond stress was highest at the embedded end as the applied load reached 44-58% and 30% of its maximum value respectively.
6. The maximum local bond stress exceeded the calculated average bond stress by a factor of about 1.3 for the water saturated specimens, 2.5 for the sealed control specimens, and 4.5 for the oil saturated specimens.

7.1.4 Dynamic Modulus of Elasticity.

1. The dynamic modulus of concrete decreased as the water-cement ratio increased.
2. The dynamic modulus of the 7-day water cured concrete was reduced by 17-18.5% after being dried in the oven for 14 days at 105° C, but upon resaturation in water, the reduction was completely recovered after 65 days.
3. The dynamic modulus of 580 days oil soaked concrete prisms increased by about 12-14% over that of the dry specimens before they were soaked in oil. The increase was almost the same for all concrete mixes regardless of the maximum size of aggregates and w/c ratios.
4. The increase of the dynamic modulus was largely dependent upon the increase in the resonant frequency which was largely dependent on the percentage of liquid (water or oil) filled voids.
5. The increase in the resonant frequency after oil soaking for 580 days was between 69-71% less than the increase after soaking similar specimens in water for the same period.
6. No deterioration could be detected by the dynamic modulus measurements over the period considered. Nevertheless, the increase in dynamic modulus upon oil soaking for 580 days was between 60-65% less than the increase after soaking similar specimens in water for the same period.
7. There appears to be no single relation which can satisfactorily predict the static-dynamic modulus relationship for concrete.

7.1.5 Damping Capacity.

1. The logarithmic decrement increased as the w/c ratio increased. The 7-days moist cured damping capacity of the 20 mm aggregate concrete was between 1.5-6.0% less than that of the 10 mm. aggregate concrete cured for the same period. The reduction was significant ($P \leqslant 0.05$) for the concrete of 0.6 w/c ratio only.
2. The variation of damping capacity was largely dependent upon the percentage of water-filled pores. The damping capacity was reduced by 74-81% due to the loss of moisture from the pores and the capillary channels after drying for 14 days in an oven at 105° C to a constant weight.
3. The damping capacity for those dry concrete specimens soaked in water or crude oil, increased very rapidly at the initial stages of soaking due to high initial penetration of absorbed liquid. The rate of increase in the damping capacity of water soaked specimens was higher than that of the oil soaked specimens due to a higher rate of water penetration compared to crude oil.
4. When the water or oil soaked specimens became almost fully saturated at about 65 and 160 days respectively, the damping capacity started to decrease with a further soaking period. At this stage the logarithmic decrement depended mostly on hydration and maturity, and the influence of water or crude oil content became less significant. The percentage reduction of the water saturated specimens was between 2-2.5 times higher than that of the oil saturated specimens.

5. There is no single relationship between the damping capacity and the dynamic modulus for concretes of different curing conditions, moisture content, and maximum size of aggregate.

7.2 LIMITATIONS OF THE PRESENT WORK.

The limitations of the present investigation are considered to be as follows:

7.2.1 Materials.

1. Only Kuwait crude oil has been used and at a constant temperature.
2. Only one type of aggregate - natural river sand, of 10 and 20 mm. maximum size, has been used.
3. Only Ordinary Portland Cement has been used.

7.2.2 Mechanical properties.

1. One standard initial curing procedure has been used for all specimens.
2. To accelerate the oil penetration, samples were first dried to 105° C before soaking in oil which may have had an adverse affect on the HCP and concrete properties although indications are that this is probably small.
3. Pulse velocity tests were done on cubes soaked in oil for 600 days only.
4. Unrestrained shrinkage study was carried out for those mixes designed primarily to be impermeable and sufficiently workable.

7.2.3 Dynamic and Damping properties.

1. In the dynamic tests, the specimens were only excited to resonance in the longitudinal mode.
2. The tests were carried out for concrete only.

7.2.4 Bond Strength.

1. Only one bar diameter of 20 mm. and embedded length of 150 mm. was used in the pull-out test and there is no evidence that these results can be extrapolated to other test conditions.
2. The bond stress distribution tests were only made on plain bars.
3. The bond stress distribution was measured for specimens soaked in oil for a period of 8 months, having only 85% degree of oil saturation.

7.3 RECOMMENDATIONS FOR FUTURE WORK.

Areas in which further work is needed for a better understanding of the effects of crude oil saturation on the structure and properties of HCPs and concrete are:

1. Long and short term properties of oil saturated concrete manufactured using different types of cement and aggregate.
2. The study of the mechanical properties of concrete soaked in oil heated to different temperature.
3. The effect of different petroleum products on the properties of concrete.
4. Study of the characteristics of the internal microcracks and crack development under load for oil saturated concrete.
5. The effect of different admixtures on the properties and impermeability of concrete in direct contact with crude oil.
6. Study of the flow of crude oil into cracks in concrete.
7. Further study of the mechanism and distribution of the bond stress between the reinforcement and concrete of oil saturated specimens.

8. Study of the structural behaviour of reinforced and prestressed oil soaked concrete members.
9. Study of the creep characteristics of oil saturated concrete.

REFERENCES

1. Bell, B.J. and Deveaux, D.R., "Why Prestressed Concrete in the Refinery". Mechanical Engineering, Dec. 1953, pp.970-978.

2. Shapard, E.R. "Concrete Gasoline Tanks for Military Use". A.C.I. Journ., Proc.Vol.40, April,1944, pp.429-438.

3. Spamer, M.A. "Navy Installation of Protective Linings for Prestressed Concrete Tanks Containing Liquid Fuels". A.C.I. Journ., Proc. Vol.40, April 1944, pp.417-428.

4. Pearson, J.C., "Tests of Gasoline-Resistant Coatings", A.C.I. Journ., Proc.Vol.40, Feb. 1944, pp.281-290.

5. Macpherson, A.R."Precast Concrete Oil Tanks" Tacoma Builders Supply Co.,Concrete Products,June,1952, pp.182-183.

6. Hampe, E., "Recent Developments in the Construction of Liquid Tanks". F.I.P. notes 67, March-April, 1977, pp.11-16.

7. Gerwick, B. and Hognestad "Concrete Oil Storage Tank Placed on North Sea Floors". A.S.C.E. Civil Engineering Journ., Aug. 1973, pp.81.

8. Richardson, C. "North Sea Oil Structure". Proc. Symp. University of Birmingham, on "Concrete-can it hold water". Sept. 1974, pp.47-54.

9. Lea, F.M. "The Chemistry of Cement and Concrete" London, Edward Arnold (Publishers) Ltd., 1970, pp.659-665.

10. Biczock, I. "Concrete Corrosion and Concrete Protection". Publishing house of the Hungarian Academy of Science, Budapest, 1964, pp.324-329.

11. Hernibrock, F.B. "The Effectiveness of Various Treatment and Coating for Concrete in Reducing Penetration of Kerosene". A.C.I. Journ., Proc. Vol.41, Sept. 1944, pp.13-20.

12. Lacaraix, R.L. "Special Problems in Connection with Under Water Oil Storage Tanks of Prestressed Concrete". F.I.P. Symp. on Concrete Sea Structure, TBILISI, Sept. 1972, pp.94.

13. Stoner, F.R. et al., "Discussion of a paper by Spamer, M.A. and a paper by Sherpard, E.R." A.C.I. Journ., Proc. Vol.40, Nov. 1944, pp.440-1 to 440-6.

14. Meissner, H. and Pearson, J. Discussion of the paper "Tests of Gasoline-Resistance Coatings". A.C.I., Proc. Vol.15, No. 6, 1944, pp.292.

15. Hansen, T.C. "Creep of Oil Saturated Concrete". Proc. Int. Conference on Mechanical Behaviour of Materials.,Kyoto, August 1971, pp.257-261.

16. Hirschithal, M. "Effect of Oils on Concrete". A.C.I. Journ., Proc. Vol.45, May 1949,pp.681.

17. Bahdadur, A.R. "Oil - Mixed Cement Mortar and Concrete". Indian Concrete Journal, Dec. 1949, pp.290.

18. Faiyadh, F.I. "Further Investigations of the Properties of Oil Soaked Concrete". M.Eng. Thesis, University of Sheffield, 1976.

19. Matti, M.A. "The Properties of Oil Soaked Concrete". M.Eng. Thesis, University of Sheffield, 1974.

20. Matti, M.A. "Some Properties and Permeability of Concrete in Direct Contact with Crude Oil". Ph.D. Thesis, University of Sheffield, 1976.

21. Oyeka, C.C. "Pore Structure and Crude Oil Permeability of Hardened Cement Paste and Concrete". Ph.D. Thesis, University of Sheffield, 1978.

22. Newman, K. "The Structure and Engineering Properties of Concrete". Proc. of an Int. Symp. on the Theory of Arch Dams, Southampton, April 1964, London, Pergamon Press Ltd., 1965, pp.683-712.

23. Hansen, T.C. "Theories of Multi-Phase Materials Applies to Concrete, Cement Mortar, and Cement Paste". The Structure of Concrete and its Behaviour Under Load, Proc. of an International Conference, London, Sept. 1965, pp.16-23.

24. Hobbs, D.W. "The Compressive Strength of Concrete, a Statistical Approach to Failure". Magazine of Concrete Research, Vol.24, No. 80, Sept. 1972, pp.127-138.

25. Newman, K. "Properties of Concrete". Structural Concrete, Vol.2, No. 11, Sept. 1965, pp.451-482.

26. Hobbs, D.W. "The Strength and Deformation of Concrete Under Short-term Loading". Technical Report, Cement and Concrete Association, Sept.1973, pp.11-17.

27. Hansen, T.C. "Creep and Stress Relaxation of Concrete a Theoretical and Experimental Investigation". Swedish Cement and Concrete Research Institute, Stockholm, Sept. 1960, pp.112.

28. Powers, T. and Brownyard,T. "Studies on the Physical Properties of Hardened Cement Pastes". Proc. A.C.I., Vol.43, 1947, pp.101, 249, 469, 549, 845, and 933.

29. Powers, T. "Structure and Physical Properties of Hardened Cement Pastes". Journ. Amer. Ceramics Soc., Vol.41, No. 1, Jan. 1958, pp.1-6.

30. Powers, T.C. "The Physical Structure and Engineering Properties of Concrete". Portl. Cement Assoc., Res. Dept., Bull. 90, Chicago, 1958.

31. Philleo, R. "The Origin of Strength of Concrete". Highway Res. Bd., Spec. Rept. 90, 1966, pp.175-185.

32. Feldman, R. and Sereda, P. "Model for Hydrated Portland Cement Paste as Deduced from Sorption-Length Changes and Mechanical Properties". Materials and Structure, Vol.1, No. 6, 1968, pp.509-520.

33. Scholer, C.F. "The Role of Mortar-Aggregate Bond in the Strength of Concrete". Highway Research, Record No. 210, 1967, pp.108-117.

34. Alexander, K.M. et al. "Aggregate-Cement Bond, Cement Paste Strength, and Strength of Concrete". The Structure of Concrete and its Behaviour Under Load, Proc. of an Int. Conf., London, Sept. 1965, pp.59-81.

35. Alexander, K.M. "Strength of Cement-Aggregate Bond". A.C.I.Journal, Proc. Vol.56, No. 5, Nov.1959, pp.377-390.

36. Cordon, W.A. and Gillespie, H.A. "Variables in Concrete Aggregates and Portland Cement Paste which Influence the Strength of Concrete". A.C.I. Journ., Proc. Vol.60, August 1963, pp.1029-1050.

37. Jones, R. and Kaplan, M.R. "The Effect of Coarse Aggregate on the Mode of Failure of Concrete in Compression and Flexural". Magazine of Concrete Research, Vol.9, No. 26, August 1957, pp.89-94.

38. Newman, K. "Criteria for the Behaviour of Plain Concrete Under Complex States of Stress". The Structure of Concrete and its Behaviour Under Load". Proc. of an Int. Conf., London, Sept.1965, pp.255-274.

39. Richart, F.E., Brandtzaeg, A. and Brown, L. "A study of the Failure of Concrete under Combined Compressive Stresses". Urbana, University of Illinois Engineering Experiment. Station, April 1929, Bulletin No. 185, pp.102.

40. Jones, R.L. "A method of Studying the Formation of Cracks in a Material Subjected to Stress". British Journal of Applied Physics, Vol.3, No. 7, July 1952, pp.229-232.

41. Slate, F.O. and Olsefski, S. "X-Rays for Study of Internal Structure and Microcracking of Concrete". A.C.I., Proc. Vol.60, No. 5, May 1963, pp.575-588.

42. Shah, S.P. and Slate, F.O. "Internal Microcracking, Mortar-Aggregate Bond and the Stress-Strain Curve of Concrete". The Structure of Concrete and its Behaviour Under Load. Proceedings of an International Conference, London, Sept. 1965, pp.82-92.

43. Jones, R. "Cracking and Failure of Concrete Test Specimens Under Uniaxial Quasi-Static Loading". The Structure of Concrete and its Behaviour Under Load". Proc. of an Inter. Conf., London, Sept. 1965, pp.125-130.

44. Newman, K. and Lachance, L. "Testing of Brittle Materials Under Uniform Uniaxial Compressive Stress". A.S.T.M., Procs. Vol.64, 1964, pp.1044-1067.

45. Taylor, M.A. "General Behaviour Theory for Cement Pastes, Mortars, and Concretes". A.C.I. Proc., Oct. 1971, pp.756-762.

46. Hansen, H. et al. "Compressive Strength of Concrete-Cube or Cylinder". Bulletin Rilem, No. 17, Dec. 1962, pp.23-30.

47. Hughes, B.P. and Bahramian, B. "Cube Tests and the Uniaxial Compressive Strength of Concrete". Magazine of Concrete Research, Vol.17, No. 53, Dec. 1965, pp.177-182.

48. Neville, A.M. "The Failure of Concrete Compression Test Specimens". Civil Engineering and Public Works Review, Vol.52, No. 613, July 1957, pp.773-774.

49. Glucklich, J. "On the Compression Failure of Plain Concrete". Urbana, University of Illinois, March, 1962, pp.31. T. & A.M. Report No.215.

50. Davies, J.D. "A Modified Splitting Test for Concrete Specimens". Magazine of Concrete Research, Vol.20, No. 64, Sept. 1968, pp.183-186.

51. Baharara, A.L. "A New Indirect Tensile Test for Concrete". Indian Concrete Journal, March 1961, pp.85-89.

52. Wright, P.J.F. "Comments on an Indirect Tensile Test on Concrete Cylinders". Magazine of Concrete Research, July 1955, Vol.7, No. 20, pp.87-96.

53. Welch, G.B. "Tensile Splitting Test on Concrete Cubes and Beams". Civil Engineering and Public Work Review, August 1965, Vol.20, No. 709, pp.1161-1167.

54. Ramakrishnan, V. et al. "The Determination of the Tensile Strength of Concrete, a Comparison of Different Methods". Indian Concrete Journal, May 1969, pp.202-206.

55. Ramesh, C.K. and Chopra, S.K. "Determination of Tensile Strength of Concrete and Mortar by Split Test". Indian Concrete Journal, Sept. 1960, Vol.34, No. 9, pp.354-357.

56. "The Flexural Strength of Plain Concrete, its Measurements and Use in Designing Concrete Mixes". Road Research Technical Paper No. 67, 1964.

57. Wright, P.J.F. and Garwood, F. "The Effect of the Method of Test on the Flexural Strength of Concrete". Magazine of Concrete Research, No. 11, 1952, pp.67-76.

58. Johnston, C.D. and Sidwell, E.H. "Testing Concrete in Tension and in Compression". Magazine of Concrete Research, Vol.20, No. 65, Dec. 1968, pp.221-228.

59. Daves, J.D. and Bose, D.K. "Stress Distribution in Splitting Tests". A.C.I. Journal, Vol.65, No. 8, Aug. 1968, pp.662-666.

60. Hughes, B.P. and Chapman, G.P. "Direct Tensile Test for Concrete Using Modern Adhesives". Bulletin Rilem, No. 26, March 1965, pp.77-80.

61. Ross, A.D. et al. "Short and Long Term Deformations of Concrete as Influenced by its Physical Stress and State". The Structure of Concrete and its Behaviour Under Load, Proc. of an Int. Conf., London, Sept. 1965, pp.407-422.

62. Pihlajavaara, S.E. "Notes on the Drying of Concrete". State of Inst. for Tech. Res. 1963, Report Series III, Building - 74,110 p.

63. Malhotra, H.L. "The effect of Temperature on the Compressive Strength of Concrete". Magazine of Concrete Research, Vol.8, No. 22, 1956, pp.85-94.

64. Pihlajavaara, S.E. "Effect of Moisture Conditions on Strength, Shrinkage and Creep of Mature Concrete". Concrete and Cement Research, Vol. 4, No. 5, 1974, pp.761-771.

65. Johnston, C.D. and Sidwell, E.H. "Influence of Drying on Strength of Concrete Specimen". A.C.I. Proc. Vol.66, Sept. 1969, pp.748-755.

66. Gardner, N.J. and Poon, S.M. "Time and Temperature Effects on Tensile, Bond and Compressive Strength". A.C.I. Journ., Proc. Vol. 73, July 1976, pp.405-409.

67. Cady, P.D. et al. "Tensile Strength Reduction of Mortar and Concrete Due to Moisture Gradients". A.C.I. Journ., Proc. Vol. 69, Nov. 1972, pp.700-705.

68. Walker, S. and Bloem, D.L. "Effect of Curing and Moisture Distribution on Measured Strength of Concrete". Proc. Highway Res. Board, Vol.36, 1957, pp.334-346.

69. Waters, T. "The Effect of Allowing Concrete to Dry Before it has Fully Cured". Magazine of Concrete Research, Vol. 7, No. 20, July 1955, pp.79-82.

70. A.C.I. Committee "Shrinkage and Creep of Concrete". A.C.I. Bibliography No. 7, Detroit, Mich., A.C.I. 1967, pp.102.

71. A.C.I. Committee "Shrinkage and Creep of Concrete". A.C.I. Bibliography No. 10, Detroit, Mich., A.C.I., 1972, pp.100.

72. Mills, R.H. "Influence of water in areas of restricted adsorption on properties of concrete". Materials and Structures, Vol. 1, No. 6, 1968, pp.553-558.

73. R.I.L.E.M. "Physical and Chemical Causes of Creep and Shrinkage of Concrete". R.I.L.E.M. Symp., Munich, April 1968, Materials and Structures, Vol.2, No. 8, pp.103-148.

74. Feldman, R.F. "Assessment of Experimental Evidence of Models of Hydrated Portland Cement". Highway Res. Board, No. 370, 1971, pp.8-24.

75. Powers, T.C. "Mechanisms of Shrinkage and Reversible Creep of Hardened Cement Paste". The Structure of Concrete and its Behaviour Under Load, Proc. of an Int. Conf., London, Sept. 1965, pp.319-344.

76. Campbell-Allan, O. "Creep and Shrinkage of Concrete as Influenced by Concrete Practice and Environment". Constructional Review, Dec.1963, pp.23-28.

77. Davis, R.E. and Kelly, J.W. "Volume Change and Plastic Flow of Concrete". Report on Significance of Tests of Concrete and Concrete Aggregate, A.S.T.M. 1943, pp.54-68.

78. L'Hermite, R.G. "Volume Changes of Concrete". 4th Int. Symp. on the Chemistry of Cement, U.S. Dept. of Commerce, National Bureau of Standards, Washington 1962, Monograph 43, Vol.II, pp.659-694.

79. Pihlajavaara, S.E. "A review of Some of the Main Results of a Research on the Aging Phenomena of Concrete: Effect of Moisture Conditions on Strength, Shrinkage and Creep of Mature Concrete". Cement and Concrete Research, Vol.4, 1974, pp.761-771.

80. Ross, A.D. "Shape, Size and Shrinkage". Concrete and Constructional Engineering, August 1944, pp.193-199.

81. Hansen, T.C. and Mattock, A.H. "Influence of Size and Shape of Member on the Shrinkage and Creep of Concrete". Proc. A.C.I. Journal, Vol. 63, Feb. 1966, pp.267-290.

82. Fulton, F.S. "A Co-ordinate Approach to the Shrinkage Testing of Concrete and Mortars". Magazine of Concrete Research, Vol.13, No. 39, Nov. 1961, pp.133-140.

83. Mears, A.R. and Hobbs, D.W. "The Effect of Mix Proportions upon the Ultimate Air Drying Shrinkage of Mortar". Magazine of Concrete Research, Vol.24, No. 79, June, 1972, pp.77-84.

84. Alexander, K.M. "Factors affecting the drying Shrinkage of Concrete". Constructional Review, Sydney, Vol.37, No. 3, March 1964, pp.15-23.

85. Neville, A.M. "Current Problems in Shrinkage of Concrete as Affected by Cement Paste". Civil Eng. and Public Works Review, Vol. 63, Nov. 1968, pp.1223-1231.

86. Mamillan, M. "Investigation of Shrinkage of Concrete". Third Int. Conf. of the Precast Concrete Industry, Stockholm, June 1960, 16 p.

87. Pickett, G. "Effect of Aggregate on Shrinkage of Concrete and a Hypothesis Concerning Shrinkage". Journal of A.C.I., January 1956, pp.581-590.

88. Washa, G.W. "Volume Changes - Significance of Tests and Properties of Concrete and Concrete Making Material". A.S.T.M., Sp. Tech. Publ. 1966 (169-A), pp.189-201.

89. Hansen, T.C. and Nielson, E.C. "Influence of Aggregate properties on Concrete Shrinkage". Proc. A.C.I. Journal, July 1965, pp.783-793.

90. Hobbs, D.W. "The Dependence of Bulk Modulus, Young's Modulus, Creep, Shrinkage and Thermal Expansion of Concrete Upon Aggregate Volume Concentration". Materials and Structures, RILEM, Vol.4, No. 20, 1971, pp.107-114.

91. Carlson, R.W. "Drying Shrinkage of Concrete as Affected by Many Factors". Proc. A.S.T.M., Vol.38, No. 2, 1938, pp.419-440.

92. Lyse, I. "The Shrinkage and Creep of Concrete". Magazine of Concrete Research, Vol.11, No. 33, Nov. 1959, pp.143-150.

93. Fulton, F.S. "Concrete Technology". A South African Handbook, Johannesburg, Portland Cement Institute, 1969, pp.276.

94. Verbeck, G.T. "Carbonation of Hydrated Portland Cement". A.S.T.M. Special Tech. Publ. No. 205, Jan. 1958, pp.17-36.

95. Thomson, W.T. "Measuring Changes in Physical Properties of Concrete by the Dynamic Method". Proc. A.S.T.M., Vol.40, 1940, pp.1113-1121.

96. Ide, J.M. "Some Dynamic Methods for Determination of Young's Modulus". Review of Scientific Instruments, Vol.6, Oct. 1935, pp.296-298.

97. Obert, L. "Sonic Methods of Determining the Modulus of Elasticity of Building Materials Under Pressure". Proc. A.S.T.M., Vol.39, 1939, pp.981-998.

98. Kesler, G.E. and Chang, T.S. "Review of Sonic Methods for Determination of Mechanical Properties of Solid Materials". A.S.T.M., Bulletin No. 225, Oct. 1957, pp.40-46.

99. Thompson, W.T. "Measuring Changes in Physical Properties of Concrete by the Dynamic Methods". Proc. A.S.T.M., Vol.40, 1940, pp.113-1129.

100. Spinner, S., and Teft, W.E. "Method for Determining Mechanical Resonance Frequencies and For Calculating Elastic Moduli from these Frequencies". Proc. A.S.T.M., Vol.61, 1961, pp.1212-1238.

101. Kolsky, H. "The Propagation of Longitudinal Elastic Waves along Cylindrical Bars". Philosophical Magazine, Vol.45, 1954, pp.712-725.

102. Pickett, G. "Equations for Computing Elastic Constants from Flexural and Torsional Resonant Frequencies of Vibration of Prisms and Cylinders". Proc. A.S.T.M., Vol.45, 1945, pp.847-865.

103. B.S.I. "Methods of Testing Concrete". B.S.1881, London, 1970.

104. A.S.T.M. "Standard Methods for Fundamental Transverse, Longitudinal and Torsional Frequencies of Concrete Specimens". A.S.T.M. Standards, C215-60, Part 10, 1968, pp.155-159.

105. Brunarski, L. "Recommendations for Use of Resonance Methods for Testing Concrete-Resonance Frequency Measurements". R.I.L.E.M. Materials and Structures, Vol.2, No. 10, July-August, 1969, pp.269-273.

106. Rayleigh , J.W. "Theory of Sound". Dover Press, New York, 2nd Edition, 1945.

107. Jones, R. "Non-Destructive Testing of Concrete". Cambridge University Press, 1962, pp.104.

108. Swamy, N. and Rigby, G. "Dynamic Modulus of Hardened Paste Mortar and Concrete". Material and Construction, Vol.4, No. 19, 1971, pp.13-39.

109. Peattie, K.R. "Tests of the Bond Between Concrete and Steel". Civil Engineering and Public Works Review, Vol.51, No. 596, Feb. 1956, pp.181-184, 314-316.

110. Plowman, J.M. "The Measurement of Bond Strength". Symp. on Bond and Crack Formation in Reinforced Concrete, Stockholm, 1957, pp.147-160.

111. Pogany, A. "Nevere Untersuchungen Uber das Wesen der Haftestigkeit". Zement, Vol. 19, 1940.

112. Houde, J., and Mirza, S. "A Study of Bond Stress-Slip Relationships in Reinforced Concrete". Structural Concrete Series No. 72-8, April 1972, McGill University, Montreal Canada.

113. Wilkins, R.J. "Some Experiments on the Load Distribution in Bond Tests". Magazine of Concrete Research, Vol. 2, No. 5, Jan. 1951, pp.65-72.

114. Ferguson, P.M. et al. "Pull-Out Tests on High Strength Reinforcing Bars". A.C.I. Proc., Vol.62, August 1965, pp.933-950.

115. Snowdon, L.C. "Classifying Reinforcing Bars for Bond Strength". B.R.S., Current Paper 36/70, Dept. of Environment, Nov. 1970, pp.32.

116. Nilson, A.H. "Bond Stress-slip Relations in Reinforced Concrete". Report 345, Dept. of Structural Engineering, School of Civil and Environmental Engineering, Cornell University, New York.

117. Watstein, D. "Bond Stress in Concrete Pull-Out Specimens". A.C.I. Journ., Proc. Vol.38, Sept. 1941, pp.37-50.

118. Glanville, W.H. "Studies in Reinforced Concrete Bond Resistance". Dept. of Scientific and Industrial Research, Building Research Technical Note No. 10, London 1930, pp. vi - 37.

119. Mains, R.M. "Measurement of the Distribution of Tensile and Bond Stresses along Reinforcing Bars". A.C.I. Journ., Proc. Vol.48, No. 3, Nov. 1951, pp.225-252.

120. Johnston, B. "The Deformation of Reinforced Concrete Beams". Ph.D. Thesis, University of Sheffield, 1973.

121. Perry, E.S., and Thompson, J.N. "Bond Stress Distribution on Reinforcing Steel in Beams and Pull-Out Specimens". A.C.I. Journ., Proc. Vol.63, No. 8, August 1966, pp.865-875.

122. Rehm, G. "The Basic Principles of the Bond Between Steel and Concrete". Translation No. 134 by C. Van Amerongen, 1961, pp.1-67.

123. Ruiz, W.H. "Effect of Repeated Loads on the Rotation Capacity of Reinforced Concrete Beams". Ph.D. Thesis, Cornell University, Sept. 1968.

124. Neville, A.M. "Properties of Concrete". Sir Isaac Pitman and Sons Ltd., 1973, London.

125. B.S.I. "Specification for Aggregates from Natural Sources for Concrete Including Granolithic". B.S.882, 1201: Part 2: 1973.

126. A.S.T.M. (C190-72). "Standard Method of Test for Tensile Strength of Hydraulic Cement Mortar". American National Standards Institute.

127. Teychenne, D. et al. "Design of Normal Concrete Mixes". Dept. of the Environment, B.R.E., HMSO, London, 1975.

128. Elvery, R.H. and Haroun, W. "A Direct Tensile Test for Concrete Under Long and Short-Term Loading". Magazine of Concrete Research, Vol.20, No. 63, June 1963, pp.111-116.

129. C.N.S. Instruments Ltd. "Pundit Manual for Use with the Portable Ultrasonic Non-Destructive Digital Indicating Tester". 61-63 Holmes Road, London.

130. Cordon, W.A. "Size and Number of Samples and Statistical Considerations in Sampling". Special Tech. Publ. STP 169, A.S.T.M., Philadelphia, 1955.

131. Davis, D. and Goldsmith, P. "Statistical Method in Research and Production". Published for I.C.I. by Oliver and Boyd, Edinburgh, 1972, pp.237-286.

132. Brunauer, S. et al. "The Stoichiometry of the hydration of Dicalcium Silicate and Tricalcium Silicate at Room Temperature". J. Amer. Chem. Soc., Vol.80, 1958, pp.761-767.

133. Shah, S. and Chandra, S. "Critical Stress, Volume Changes, and Microcracking of Concrete". Journal A.C.I., Proc. Vol. 65, 1968, pp.770-781.

134. Newman, K. and Newman, J.B. "Failure Theories and Design Criteria for Plain Concrete". Structure, Solid Mechanics and Engineering Design. The Proceedings of the Southampton 1969 Civil Engineering Materials Conference. Part 2, pp.963-995.

135. Fagerlund, G. "Elastic Modulus of Concrete". IVPAC/RILEM, Int. Symp. on Pore Structures and Properties of Materials, Vol. II, Prague 1973, pp. D129-149.

136. Fagerlund, G. "Strength and Porosity of Concrete". IVPAC/RILEM, Int. Symp. on Pore Structure and Properties of Materials, Vol. V, Prague 1973, pp.D51-73.

137. Ori-Ishai. "The Time-Dependent Deformation Behaviour of Cement Paste, Mortar and Concrete". The Structure of Concrete and its Behaviour Under Load. Proc. of an Int. Conf., London, Sept. 1965, pp.345-364.

138. Neville, A.M. "Theories of Creep in Concrete". A.C.I. Journal, Proc. Vol.52, No. 1, Sept. 1955, pp.47-60.

139. Hansen, J.A. "Effects of Curing and Drying Environments on Splitting Tensile Strength of Concrete". A.C.I. Proc., Vol.65, July 1968, pp.535-543.

140. Van Eeckhout, E.M. "The Mechanisms of Strength Reduction Due to Moisture in Coal Mine Shales". Int. Journal of Rock Mech., Min. Sci., Vol. 13, pp.61-67.

141. Rehbinder, P. and Lichtman, V. "Effect of Surface Active Media on Strains and Rupture in Solids". Proceedings of the Second Int. Conf. on Surface Activity, Vol.3, Butterworth Scientific Publications, London, 1957, pp.563-580.

142. Colback, P.S.B. and Wiid, B.L. "The Influence of Moisture Content on the Compressive Strength of Rock". Proc. Rock Mech. Symp., Toronto, 1965, pp.65-83.

143. Hernnikoff, A.J. "Shrinkage, Swelling, and Creep in Cement". Eng. Mech. Div., Proc. of the A.S.C.E., Vol.85, 1959, pp.115-135.

144. Cook, D. and Hague, M. "Strength Reduction and Length Changes in Concrete and Mortar on Water and Methanol Sorption". Cem. and Conc. Res., Vol. 4, 1974, pp.735-744.

145. Hannant, D.J. "The Mechanics of Creep in Concrete". Material and Structure, Vol.1, No. 5, 1968, pp.403-410.

146. Facaoaru, I. et al. "Studies on the Deformation and Cracking of Concrete Under Uniaxial Stress Fields". Structure, Solid, Mechanics and Engineering Design, The Proc. of the Southampton 1969, pp.653-665.

147. Alexander, K.M., Wardlow, J. "Dependence of Cement-Aggregate Bond Strength on Size of Aggregate". Nature, Vol.187, July 1960, pp.230-231.

148. Powers, T.C. "Causes and Control of Volume Change". Portland Cement Association, Journal of Research and Development lab., Vol.1, No. 1, Jan. 1959, pp.29-39.

149. Glocklich, J. and Ishai,O. "Creep Mechanisms in Cement Mortar". A.C.I. Journ., Vol.59, No. 7, July 1962, pp.923-941.

150. White, A.H. "Volume Changes of Portland Cement Affected by Chemical Composition and Aging". Proc., ASTM, Vol.28, 1928, pp.398-421.

151. Billing, K. "Structural Concrete". MacMillan & Co. Ltd., London, 1963.

152. Bennett, E.W., and Khilji, Z.M. "The Effect of Some Properties of the Coarse Aggregate in Hardened Cement". Jour. of the British Granite and Whinstone Federation, Vol.3, No. 2, Autumn 1963, pp.17-28.

153. B.S.I. "CP110: Code of Practice for the Structural Use of Concrete, Part 1, Design, Materials and Workmanship". 1972.

154. Takayayashi, M.T. "Comparison of Dynamic Young's Modulus and Static Young's Modulus for Concrete". RILEM International Symposium on Non-destructive Testing of Materials and Structures, Paris 1953, Vol.1, No. A-3, pp.34-44.

155. B.S.I. "Specification for Hot Rolled Steel Bars for the Reinforcement of Concrete". B.S.I. 4449, 1969.

156. Davis, R.E. and Troxel, B.E. "Modulus of elasticity and Poisson's ratio for concrete and the influence of age and other factors upon these values". Proc. of the A.S.T.M., Vol.29, 1929, pp.678-710.

157. Sereda, P.I. et al. "Effect of sorbed water on some mechanical properties of hydrated cement pastes and compacts". Highway Research Board, Special Report, No. 90, 1966, pp.58-73.

158. Johnston, C.D. "Concrete and its constituent materials in uniaxial tension and compression". Ph.D. Thesis, Queens University of Belfast, 1967.

159. Hague, M.N., and Cook, D.J. "The effect of water sorption on the dynamic modulus of elasticity of desicated concrete materials". Material and Construction, Vol. 9, No. 54, Dec. 1976, pp.407-410.

160. Cole, D.B. "The Damping Capacity of Hardened Cement Paste, Mortar, and Concrete Specimens". Proc. of Sym. on Vibration in Civil Eng., Butterworth Scientific Publication, London, 1966, p.235-247.

161. Hansen, T.C. "Influence of aggregate and voids on modulus of Elasticity of Concrete, Cement Mortar and Cement Paste". Jour. A.C.I., No. 62, Feb. 1965, pp.193-216.

162. Klieger, P. "Long-Time Study of Cement Performance in Concrete". Journ. of A.C.I., Proc. Vol. 54, No. 6, Dec. 1957, pp.481-504.

163. Stanton, T.E. "Tests Comparing the Modulus of Elasticity of Portland Cement Concrete as Determined by the Dynamic and Compression Methods". ASTM Bulletin,No. 131, Dec., 1944, pp.17-20.

164. Elvery, R.H. and Evans, E.P. "The effect of curing conditions on the physical properties of concrete". Magazine of Concrete Research, Vol.16, No. 46, March 1964, pp.11-20.

165. Simmons, J.C. "Poisson's Ratio of Concrete - a comparison of dynamic and static measurements". Magazine of Concrete Research, Vol.7, No. 20, July 1955, pp.61-68.

166. Sharma, M.R. and Gupta, B.L. "Sonic Modulus as Related to Strength and Static Modulus of High Strength Concrete". Indian Concrete Journal, Vol.34, No. 4, April 1960, pp.139-141.

167. Shrivastana, J.P. and Sen, B. "Factors Affecting Resonant Frequency and Compressive Strength of Concrete". Indian Concrete Journal, Vol.37, No. 1, Jan. 1963, pp.27-31.

168. Jones, R. and Welch, G.B. "The damping properties of plain concrete, Effect of Composition and Relations with Elasticity and Strength". Road Research Laboratory, Report LR111, 1967, pp.16.