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DEVELOPMENT OF DIMENSIONLESS HYDROGRAPHS FOR STROM SEWERS USING KINEMATIC WAVE ROUTING TECHNIQUE

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ABSTRACT

The study presents a methodology for the estimation of flood wave characteristics in rectangular sewers at a downstream location of a given inflow site using kinematic wave routing technique. For the development of dimensionless hydrographs and other relationship it is presumed that the flood wave would be contained within the sewer. Using the kinematic wave routing technique the dimensionless hydrographs are developed for rectangular sewers for different widths varying from 6 to 18 ft, different sewer slopes varying from 0.001 to 0.009 and for different inflow hydrographs shapes. The relationship have been developed between sewer slopes and time lag of peak flow at downstream location of given site for different widths of sewer. These relationship have been established for different shape of inflow hydrograph with peak flow varying from 100 to 500 cusecs. The relationship between peak flow and time lag of peak flow for given reach length corresponding to sewer slope of 0.001, and the relationship between sewer slopes, widths, peak flow and time lag of peak flow of routed hydrograph have also been established for different size of sewer for three different inflow hydrograph shapes. These relationships can be used for determining quickly the lag time of peak flow of outflow hydrograph at downstream site of the inlet point of channel, which may be useful for the economic design of size of main storm sewers.

1.0 INTRODUCTION

Storm drainage is an important aspect of Municipal Planning, both for new urbanising areas and for previously developed areas where drainage systems are inadequate. In our country studies to evaluate current drainage system or to design new systems are generally quite unsophisticated. Often emperical methods such as Rational Method is used for determining runoff from a urbanising watershed along with Manning's equation applied to steady flow condition for the computation of velocity in the conveyance system for evaluating the time of concentration of flow in the drains. These approximate methods can lead to poorly designed systems. Rapid urbanisation of river basins in and around metropolitan areas in advanced countries has forced land and water resources planners and hydrologists to develop a variety of methods for analysing problem in urban hydrology. Problem involving both design and management decisions are often so complex as to require application of mathematical models.

It is important to develop the best representation of the actual runoff situation when analysing urban water runoff problems. It would be desirable for modelling technique to be able to reproduce the non-linear runoff characteristics rather than being limited to linear responses such as those developed based on unit hydrograph technique.

Storm sewer flow are often considered as open channel flow. The flow in sewers are controlled by gravity since pressurised flow often cause problems such as pollution through leaky joints (Overton and Meadows, 1976). Open channel flow is described by one-dimensional St.Venant's equations consisting of continuity and momentum equations. Often many of the terms involved in the momentum equation are of minor importance when compared with the magnitude of bed slope term. The Kinematic wave approximation of the St.Venant's equation is based on the concept that the pressure term and the acceleration terms of the St. Venant's equations are of negligible magnitude when compared with the bed slope term. Storm sewers which are artificially constructed generally have steep slope. thus enabling the application of Kinematic wave technique for routing storm flow.

Further the Kinematic theory offers the benefits of non-linear response without needing an unduly complicated or costly solution procedure. It relates the channel and flow characteristics directly with the routing parameters. As the pressure term and the acceleration terms are absent in Kinematic wave equation, the diffusive property of the flood wave is absent and hence kinematic wave do not attenuate as most often experienced in storm sewer flow.

In the development of urban area, it is traditional to design and construct sewer systems which would allow for a minimum of surface storage by quickly collecting and discharging the water away from the property. Such action of collection and dumping the storm water of the upstream area creates drainage congestion in the downstream area as the peak storm sewer flow of the downstream drainage area may coincide with the disposed peak flow of the upstream locality. In order to avoid such a coincidence of peak flows along the main storm sewers with that of the joining branch sewers for the purpose of smaller size drain construction, it is necessary to understand the peak flow travel time and the related storm flow hydrograph characteristics.

Further, the time lag of flood peak from the inlet point to the outlet point of the sewer is a needed component in the determination of the time of concentration of the urban catchment required in the application of Rational Method.

A generalized parametric study on such flood wave characteristics is almost not existent or scarce. Assuming that the sewers are steep enough not to attenuate the flood peak, this preliminary study attempts to investigate the Kinematic wave properties of storm sewer flow in rectangular channels by developing non-dimensional flood hydrographs and the related parameter relationships. These relationships and non-dimensional flood hydrographs would be much useful at the planning and design stages of storm for the quick estimation of time lag of the outflow hydrograph provided the inflow hydrograph and channel characteristics are known. However, this study is of preliminary nature for verifying the proposed methodology of non-dimensionalising the routed hydrographs for certain inflow hydrograph shapes and for varying channel characteristics. In order to make such a study directly applicable for storm sewer design, one has to carry out a detailed study involving different sewer cross sectional shapes and inflow hydrograph shapes.

2.0 REVIEW

St. Venant's Equations

Unsteady, spatially varied flow in a rough, fixed bed open channel of arbitrary form and alignment can be described by partial differential equations for the conservation of mass and momentum. These equations were first proposed by St.Venant in 1871 and are, therefore, usually referred to as the St.Venant equations. The St. Venant equations for gradually varied, unsteady channel flow without lateral inflow are described as:

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = 0 \qquad (1)$$

and

$$\frac{\partial y}{\partial x} + \frac{V}{g}\frac{\partial V}{\partial x} + \frac{1}{g}\frac{\partial V}{\partial t} = S_o - S_f \qquad (2)$$

in which,

Q, A, y, V, g, S_o and S_f are discharge, area, depth velocity, acceleration due to gravity, bed slope and friction slope respectively. Equation (1) and (2) are respectively known as continuity and momentum equations. The first term on the left hand side of eqn.(2) denotes the variation of pressure force along the channel, the second term denotes the acceleration gradient term along the space and the third term represents the local acceleration gradient due to unsteady flow effect. The first term on the right hand side of eqn(2) represents the channel bed slope and the second term represents the friction slope.

Characteristic of Flood Waves in Channels

The St.Venant equations describe dynamic waves. Such waves propagate along two systems of characteristics: in downstream direction with speed $C_d = v + \sqrt{gy}$ and in upstream direction with speed $C_{d'} = v - \sqrt{gy}$ both measured relative to the channel bank. For subcritical and slightly superistical flow dynamic waves attenuate (Henderson, 1966).

Lighthill and Whitham (1955) have proved the existence of another type of wave with properties following principally from the equation of continuity. They termed them "Kinematic Waves". The implications of the Kinematic wave approximation can perhaps best be understood by comparing the magnitude of different terms in the momentum equation.

Henderson (1966) showed that for a uniform channel the magnitude of the pressure term depends on the steepness of the inflow hydrograph and is inversely proportional to ($S_o^{2/3}$). The two acceleration terms are both of equal order of magnitude but smaller than the pressure term, the ratio being in the order of F^2. Henderson (1966) quoted for a fast rising flood in a river in steep alluvial country the following typical absolute values of the individual terms (in ft/mile):

S_o	$\partial y / \partial x$	$\frac{V}{g} \frac{\partial V}{\partial x}$	$\frac{1}{g} \frac{\partial V}{\partial t}$
26	0.5	0.125-0.25	0.05

He also indicated that for flat bed slopes the pressure term might be of the same order as S_o.

The general conclusions from this order of magnitude analysis are

a) for steep bed slopes the gravity and friction forces are dominant. The steep bed slope corresponds to the value of So > 0.001 (Lee and Mays 1986).

b) the pressure term may be significant on mild slopes and/or for steeply using hydrographs

c) the acceleration terms are not significant unless the hydrograph includes sharp rises or falls.

Rating Curve for Kinematic Wave Flow:

For hydrologic applications the momentum equation is often expressed in the form of a rating equation, a combination of momentum equation and a flow equation such as the chezy or Mannings formula:

$$Q = CAR^m (S_f)^{1/2} \qquad \ldots(3)$$

For normal flow or quasi steady flow condition the flow formula gives

$$Q_n = CAR^m (S_o)^{1/2} \qquad \ldots(4)$$

The combination of the last two equations yields:

$$Q = Q_n \left(\frac{S_f}{S_o} \right)^{1/2} \qquad \ldots(5)$$

When there is no lateral flow the momentum equation may be rewritten in terms of S_f as

$$S_f = S_o - \frac{\partial y}{\partial x} - \frac{V}{g} \frac{\partial V}{\partial x} - \frac{1}{g} \frac{\partial V}{\partial t} \qquad \ldots(6)$$

Substitution of eqn. (6) in the expression for Q yields, a general expression for the rating curve as:

$$Q = Q_n \sqrt{1 - \frac{1}{S_o}\left(\frac{\partial y}{\partial x} + \frac{V}{g}\frac{\partial V}{\partial x} + \frac{1}{g}\frac{\partial V}{\partial t}\right)} \quad \ldots(7)$$

For steep bed slopes of channels, the variable terms within the square root are negligible in magnitude yielding the discharge relationship as:

$$Q = Q_n \quad \ldots(8)$$

i.e. the discharge at any section during unsteady flow is same as the normal discharge corresponding to any depth of flow. Therefore the St.Venant's equations governing Kinematic wave flow in channel for no lateral flow condition is reduced to:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = 0$$

and

$$Q = Q_n$$

The above eqn. clearly demonstrates the existence of one to one relationship between stage and discharge during unsteady flow. Therefore the continuity eqn. is modified as:

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial Q} \cdot \frac{\partial Q}{\partial t} = 0 \quad \ldots(9)$$

But $\frac{\partial Q}{\partial A}$ = C, the wave celerity of the Kinematic wave. Therefore the Kinematic wave equation is written as: .

$$\frac{\partial Q}{\partial t} + C\frac{\partial Q}{\partial x} = 0 \quad \ldots(10)$$

The above eqn. is a quasi linear first order partial differential equation. It is quasi linear because the Kinematic wave celerity is a function of the mean flow velocity, which increases with discharge. Since it is a first order equation it can only describe convection (wave travel) and not diffusion (wave attenuation) Its quasilinear property enables the deformation (wave skewness) of the hydrograph. Therefore the peak discharge does not get attenuated as the flood wave travels downstream of the channel. This is the important property of Kinematic wave.

Lighthill and Whitham (1955) have shown that for Froude number F < 2 Kinematic waves predominant over dynamic waves. Overton and Meadows (1976) state that kinematic wave approximation has been proven to be an accurate and efficient method of Simulating storm water runoff from small basins for both overland flow and stream channel routing.

HEC-1 flood hydrograph package of U.S.Army Corps of Engineers has the option for using Kinematic flow modelling in channels. However the numerical solution of the kinematic wave equation adopted in HEC-1 introduces numerical diffusion and thus leading to artificial attenuation of the flood peak. An extensive study on the HEC-1 numerical solution of kinematic wave equation has been made by Hromadaka and Devries (1988) who discuss at length the artificial numerical diffusion introduced by the solution procedure.

3.0 STATEMENT OF THE PROBLEM

The objective of this study which is of preliminary value, is to understand the characteristics of the kinematic wave movement in storm severs having rectangular cross-section by relating the lag time of the peak discharge of the outflow hydrograph with the corresponding peak discharges, slope and width of the channel of a given reach length.

4.0 METHODOLOGY

The study was initiated using HEC-1 option of Kinematic wave routing. But during the course of the study it was found that the algorithm adopted in HEC-1 Flood Hydrograph package is not appropriate for such a study. The reason being that HEC-1 algorithm for kinematic wave routing produced significant numerical errors, and artificial attenuation of the flood wave due to unconditional diffusion. The same has been confirmed by Hromadka and Devries (1988). Keeping this in view it was considered appropriate to study the kinematic characteristics for given inflow hydrographs, channel widths and slopes using the finite difference solution of the St.Venant's equations.

It is also seen that when the bed slope is steep, the St.Venant's solution would exhibit kinematic wave characteristics.

4.1 Governing Equations

Assuming no lateral flow, the movement of the flood wave in a channel reach is described by the St.Venant' equations, represented by the continuity equation

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = 0 \qquad \ldots(11)$$

and the momentum equation

$$\frac{\partial y}{\partial x} + \frac{V}{g}\frac{\partial V}{\partial x} + \frac{1}{g}\frac{\partial V}{\partial t} = S_o - S_f \qquad \ldots(12)$$

where the notations of the variables are as explained before.

4.2 Channel Routing:

Since the study is of preliminary nature only one reach of 4000 ft. was considered for all routing studies. The Manning's roughness coefficient for concrete sewers was considered as 0.014. The bed slopes used in the study were $S_o > 0.001$ as with these slopes the kinematic flood wave behaviour is exhibited (Lee and Mays, 1986).

4.3 Development of Dimensionless Hydrograph and Parameter Relationships

According to Kundzewicz (1986) the best approach for such type of studies is to use hypothetical inflow-outflow hydrographs so that the relationships established for the understanding of the kinematic wave characteristics are not masked by data errors. Therefore the given inflow hydrographs defined by a mathematical function were routed through the storm sewers of rectangular cross-section with different widths and slopes, using the St.Venant's equations solutions. The inflow hydrograph was in the form of Pearson type-III function as adopted by Weinmann (1977) as:

$$Q(t) = Q_o + (Q_p - Q_o)\left(\frac{t}{t_p}\right)^{\frac{1}{r-1}} \exp\left[\left(1 - \frac{t}{t_p}\right)/(r-1)\right] \quad ..(13)$$

where,

Q_o = base flow

Q_p = peak flow

t_p = time to peak

and r = the shape factor

Three different inflow hydrograph with shape factors 1.15, 1.50 and 1.75 were used for routing studies along the storm sewers with widths W=6 ft., 10 ft. and 18 ft. for the reach length of 4000 ft. The bed slopes of the channels studied were varying from 0.001 to 0.009. Using the St.Venant's solutions of these studies, the following relationships were established.

i) Dimensionless hydrograph at the downstream end of the 4000 ft. reach length relating time non-dimensionalised with reference to time to peak of the routed hydrograph and the discharge non-dimensionalised with reference to peak discharge.

ii) Relationship between bed slope S_o and time lag of peak discharge T_L of the routed hydrograph for different peak discharges.

iii) Relationship between bed slope S_o, width of the channel W, peak flow Q_p and time lag of peak discharge T_L of the routed hydrograph.

4.4 Channel Configuration and Flow Resistance Properties:

The routing was carried out in rectangular channels for the specific widths of 6ft., 10ft. and 18 ft. and for different bed slopes of 0.001 to 0.009 varying at the interval of 0.001. The roughness coefficient of the storm sewer was considered as 0.014 assuming it as a concrete sewer. As this study is of preliminary nature in developing the non-dimensional properties of kinematic wave, all the routing experiments were carried out only for a length of 4000 ft.

4.5 Peak Flow Used:

The inflow hydrographs whose form has been defined by eqn.(13) was used in this study with peak flows values 100, 200, 300 and 400 and 500 cusecs.

5.0 ANALYSIS

This section describes the analysis carried out for routing the inflow hydrograph and finding the lag time of peak flow of the routed hydrograph at the outlet of 4000 ft. reach of the rectangular cross-section storm sewers for different bed slopes and for different shapes of inflow hydrographs with γ = 1.15, 1.50 and 1.75. The analysis also describes the establishment of relationship between lag time of peak discharge of the routed hydrograph with bed slope, with peak discharge and combinedly with bed slope, peak discharge and channel width.

5.1 Routing of Inflow Hydrograph:

The hypothetical inflow hydrographs were routed in the specified channels with constant bed slope. For each channel with specified width, the slopes were varied between 0.001 to 0.009, the range in which only the kinematic behaviour of flood wave is exhibited. As this study was carried out with the purpose of understanding the kinematic behaviour of the flood wave in storm sewers using hypothetical inflow hydrograph, it was considered essential to use an accurate method of flood routing so that one is confident that no error of numerical scheme is incorporated in the arrived solution. Accordingly it was decided to use explicit scheme solution of St.Venant's equations to estimate the routed hydrograph.

5.2 Dimensionless Hydrographs

Some of the typical dimensionless hydrographs which relate time non-dimensionalised with reference to time to peak flow t_p, and discharge Q non-dimensionalised with reference to Q_p are shown in figures (1) to (6) for three different inflow hydrographs studies.

5.3 Relationship between Bed slope and Time Lag of Routed Hydrograph Peak for different Peak Flows:

The time lag of the routed hydrograph is related to the bed slopes So for a given channel width and inflow hydro graph with varying peak discharges as:

$$T_L = f\ (S_o) \qquad \ldots(14)$$

Figures (7) - (15) describe such relationships in log domain.

5.3 Relationship between Peak Flow and Time Lag of the Routed Hydrograph Peak:

The time lag of the routed hydrograph peak for a given inflow hydrograph shape is specifically related with the peak discharges studies as:

$$T_L = f(Q_p) \qquad \ldots(15)$$

such a relationship would reveal the variation between T_L and Q_P for different hydrograph shape and for specific bed slope and channel width. Such typical relationships are shown in figures (16)-(21).

5.5 Relationship between Bed slope S_o, width of channel W, Peak flow Q_p, and time lag of peak discharge T_L of the Routed Hydrograph

Although the variation of T_L with reference to bed slope S_o and peak discharge Q_p were independently studied as stated above, it is also required to arrive at the variation of T_L combinedly with that of S_o, Q_p and W as follows:

$$T_L = f(Q_p, S_o, W) \qquad \ldots(16)$$

An exponential relationship was attempted as:

$$T_L = a_3 \, Q_p^{b_3} \, S_o^{c_3} \, W^{d_3} \qquad \ldots(17)$$

where,

a_3, b_3, c_3 and d_3 are constants.

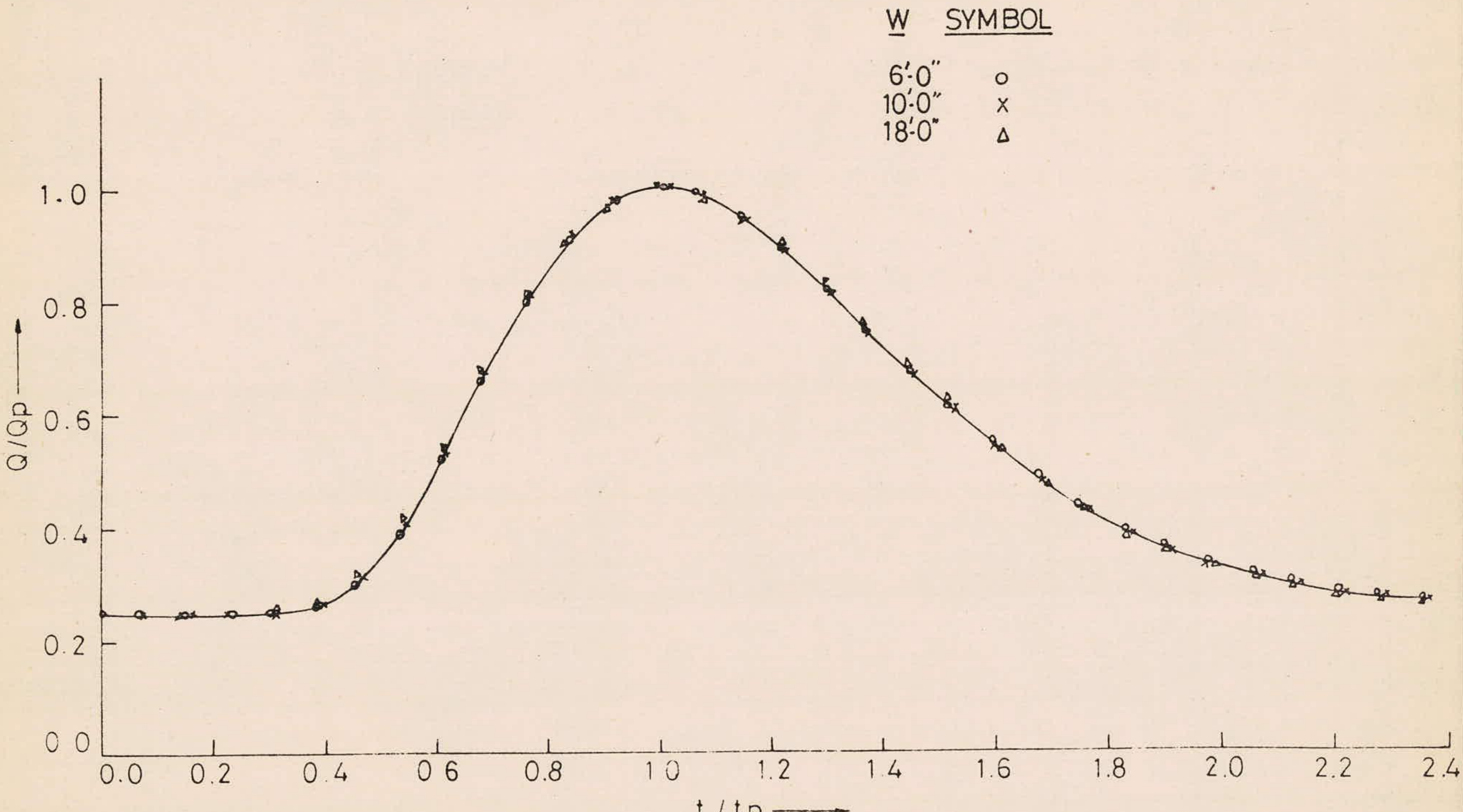


Fig. 1 DIMENSIONLESS HYDROGRAPH WITH So = 0 003 AND r =1 15

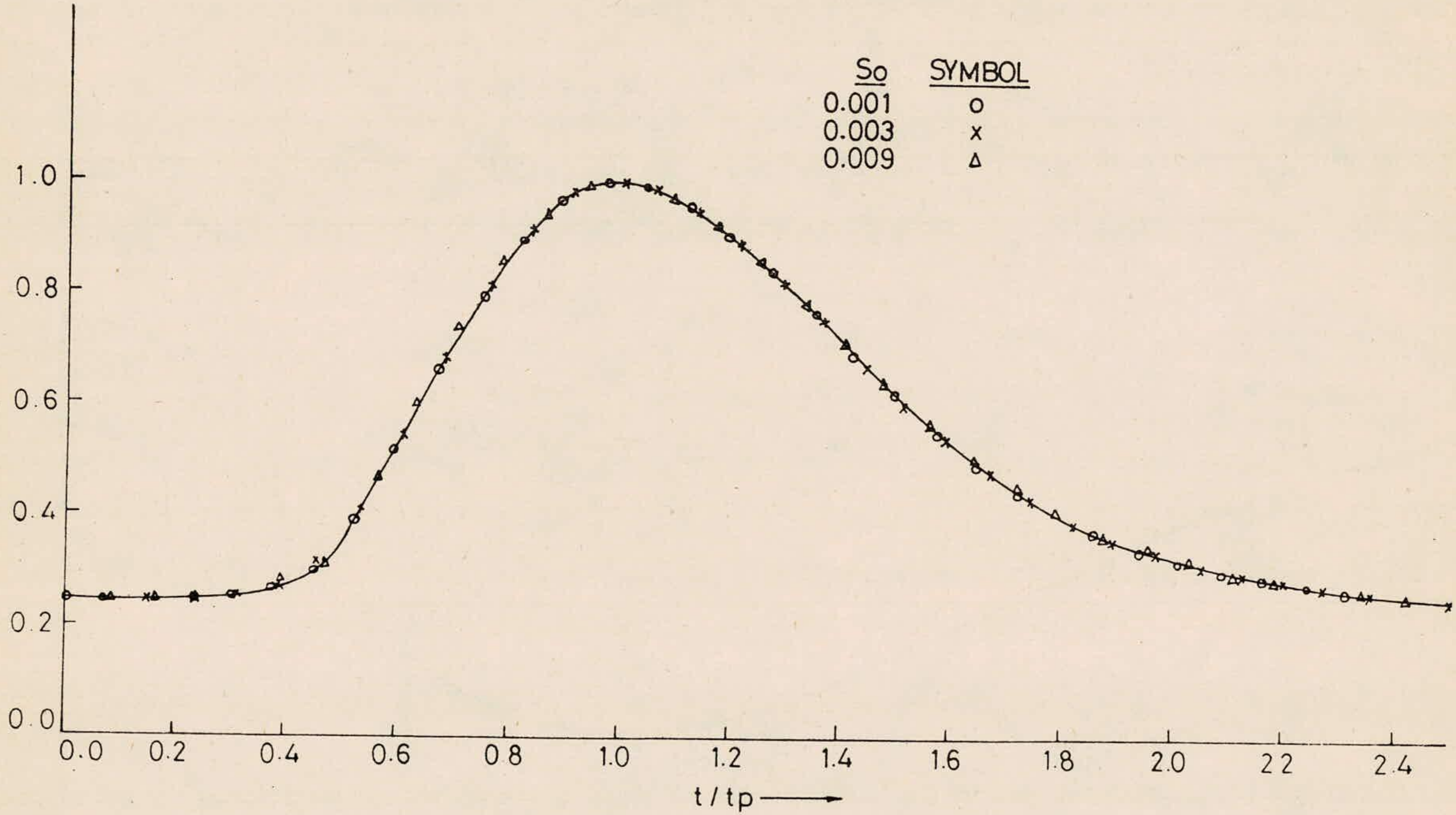


Fig. 2 DIMENSIONLESS HYDROGRAPH WITH W = 10'-0" AND r = 1 15

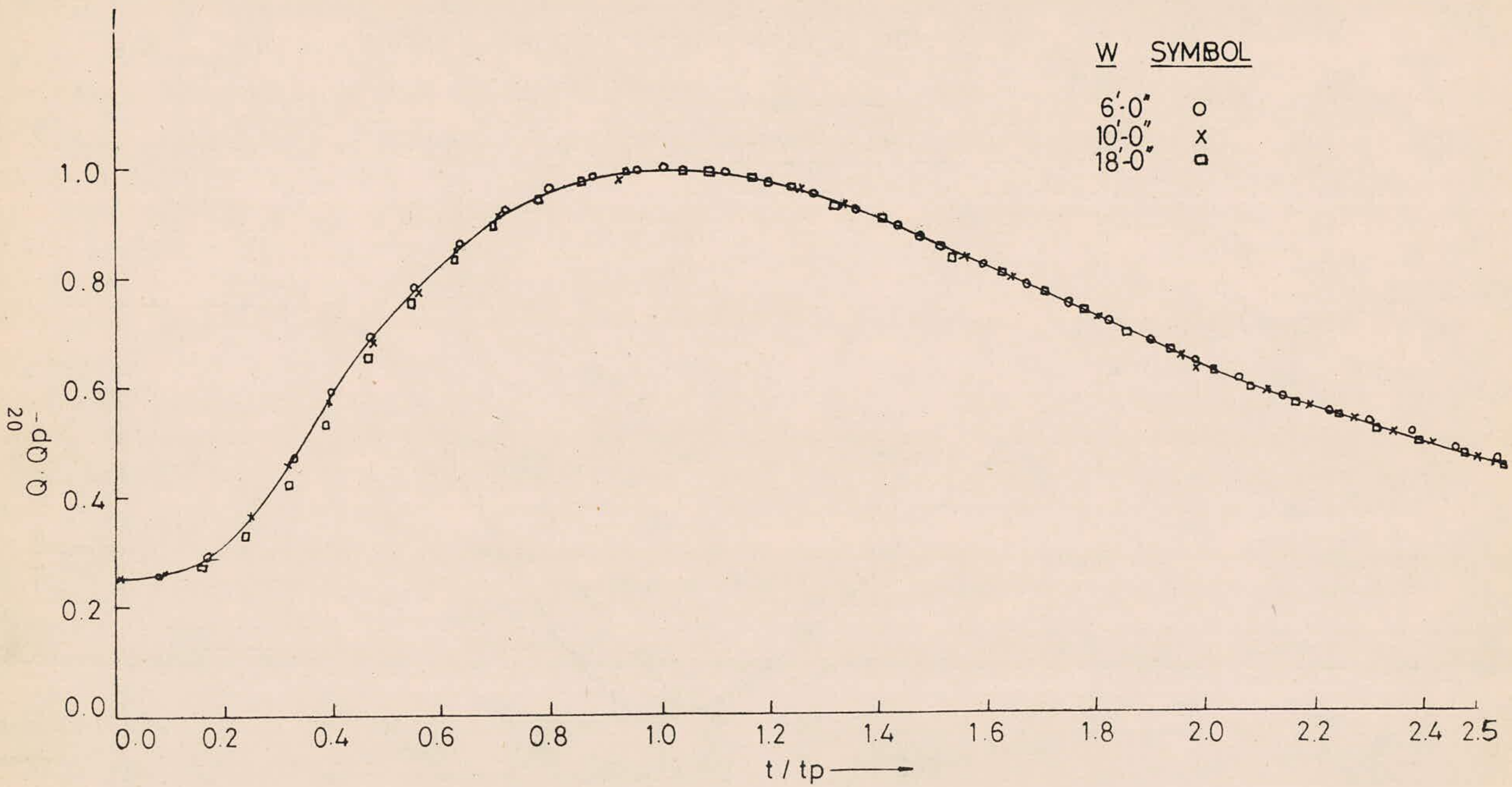


Fig. 3. DIMENSIONLESS HYDROGRAPH WITH So = 0.003 AND r = 1.50

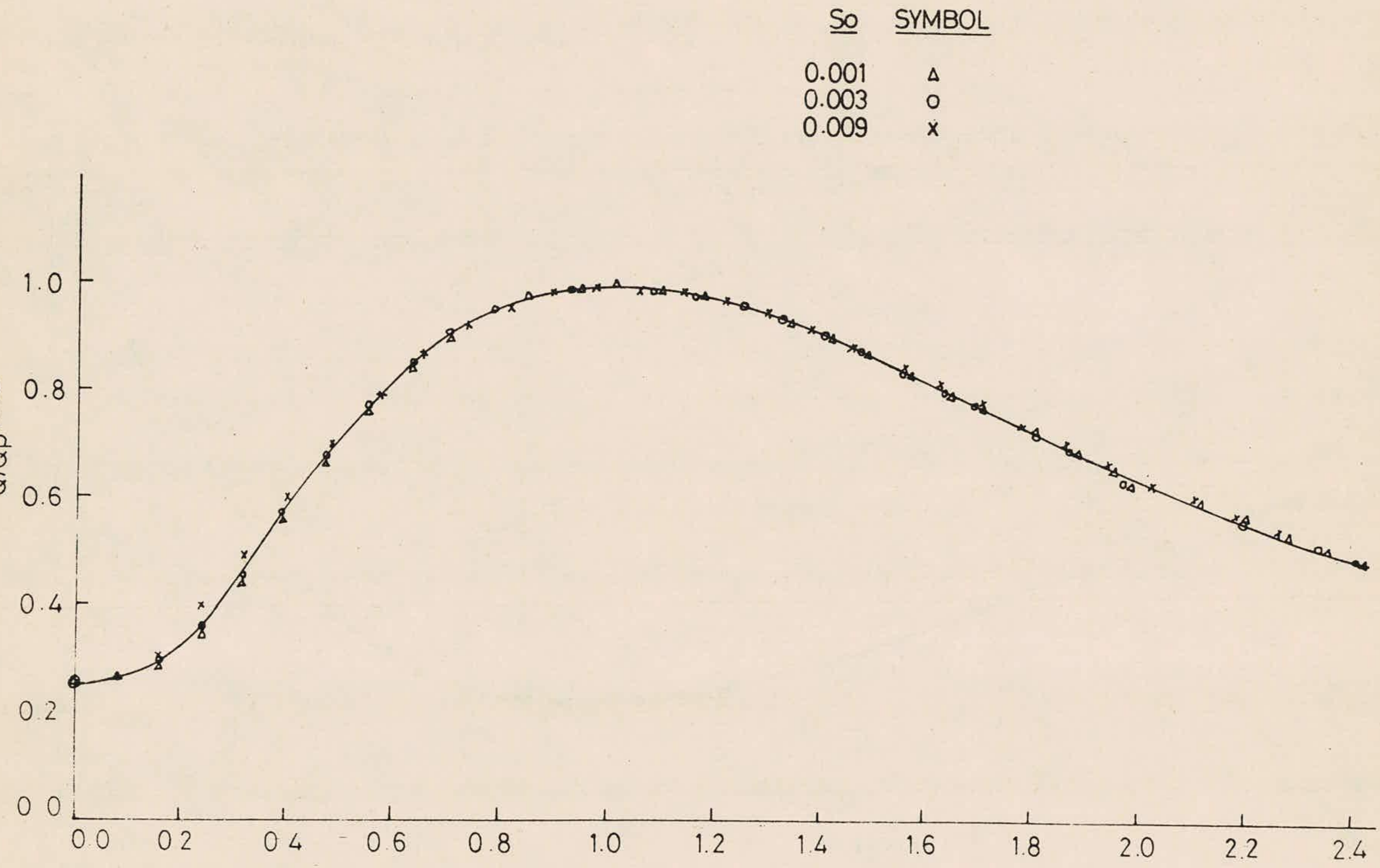


Fig. 4. DIMENSIONLESS HYDROGRAPH WITH W = 10'-0" AND r 1.50

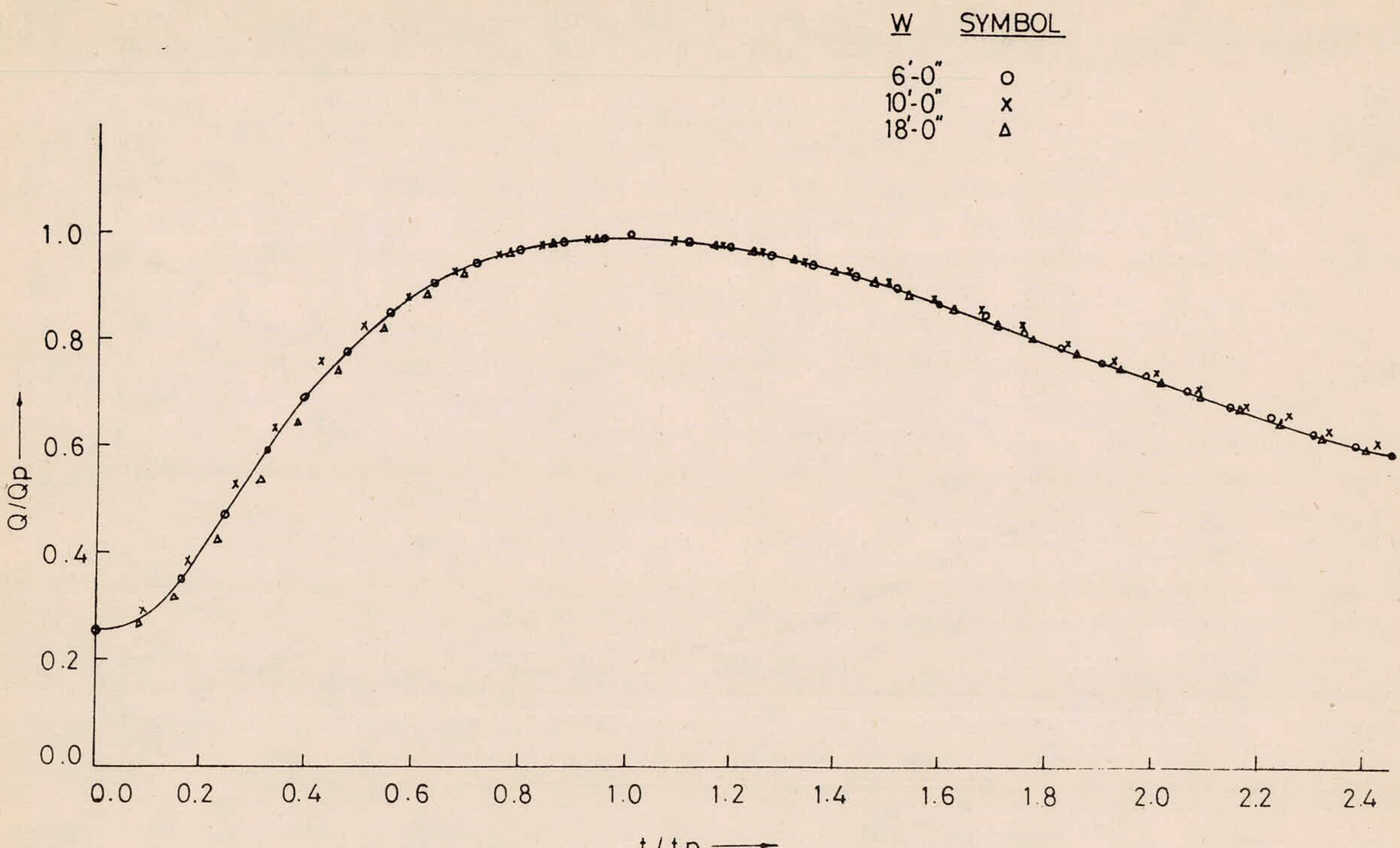


Fig.5. DIMENSIONLESS HYDROGRAPH FOR So = 0.003 AND r = 1.75

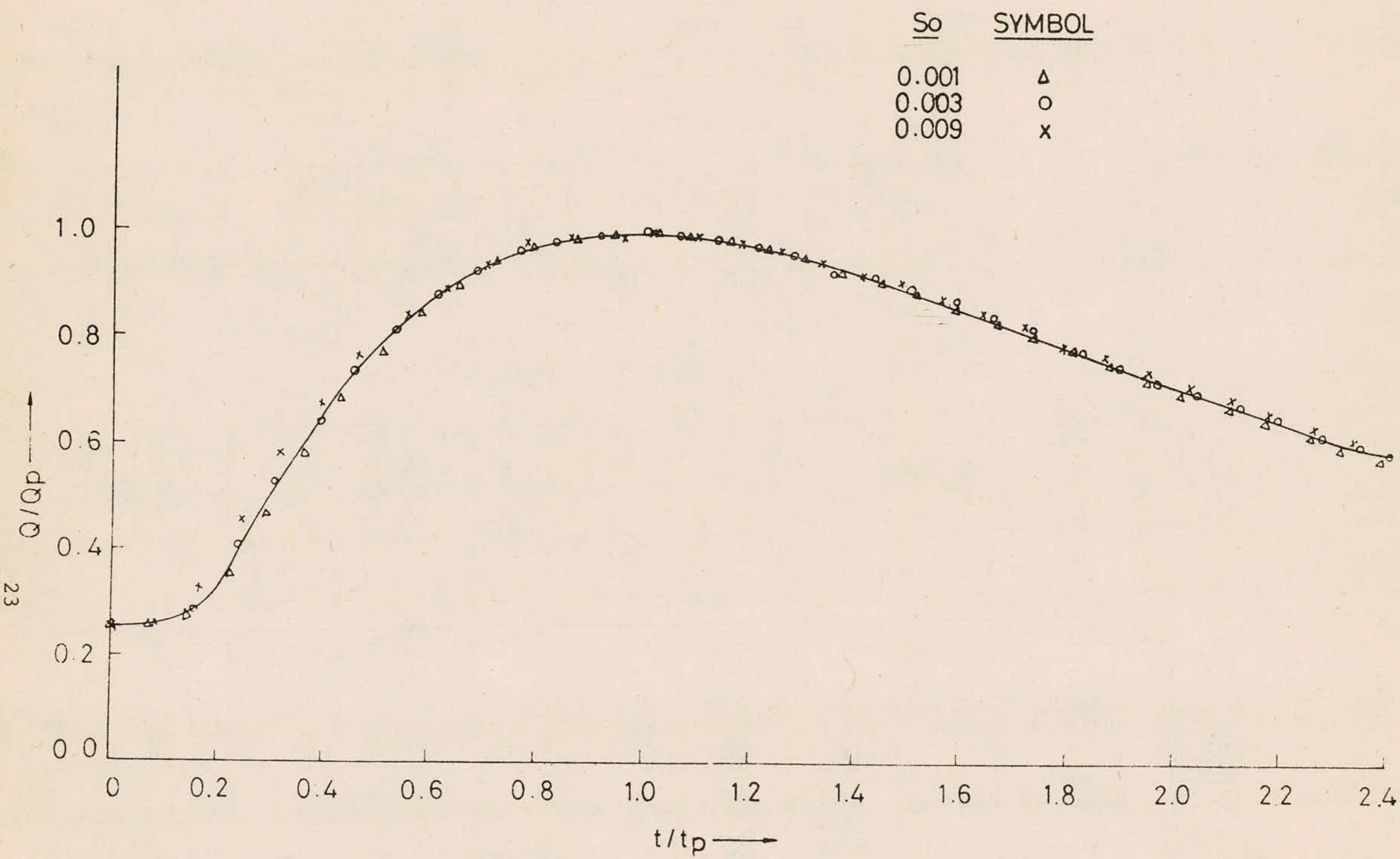


Fig.6 DIMENSIONLESS HYDROGRAPH WITH W = 10'0" AND r = 1.75

$Ln(T_L) = A - B\ Ln(So)$

CASE	Qp	SYMBOL	A	B	Cr
1	100	□	3.9010	0.36679	0.9993
2	200	x	3.6370	0.37907	0.9995
3	300	o	3.3826	0.40619	0.9987
4	400	Δ	3.4963	0.37516	0.9984
5	500	+	3.4135	0.38212	0.9937

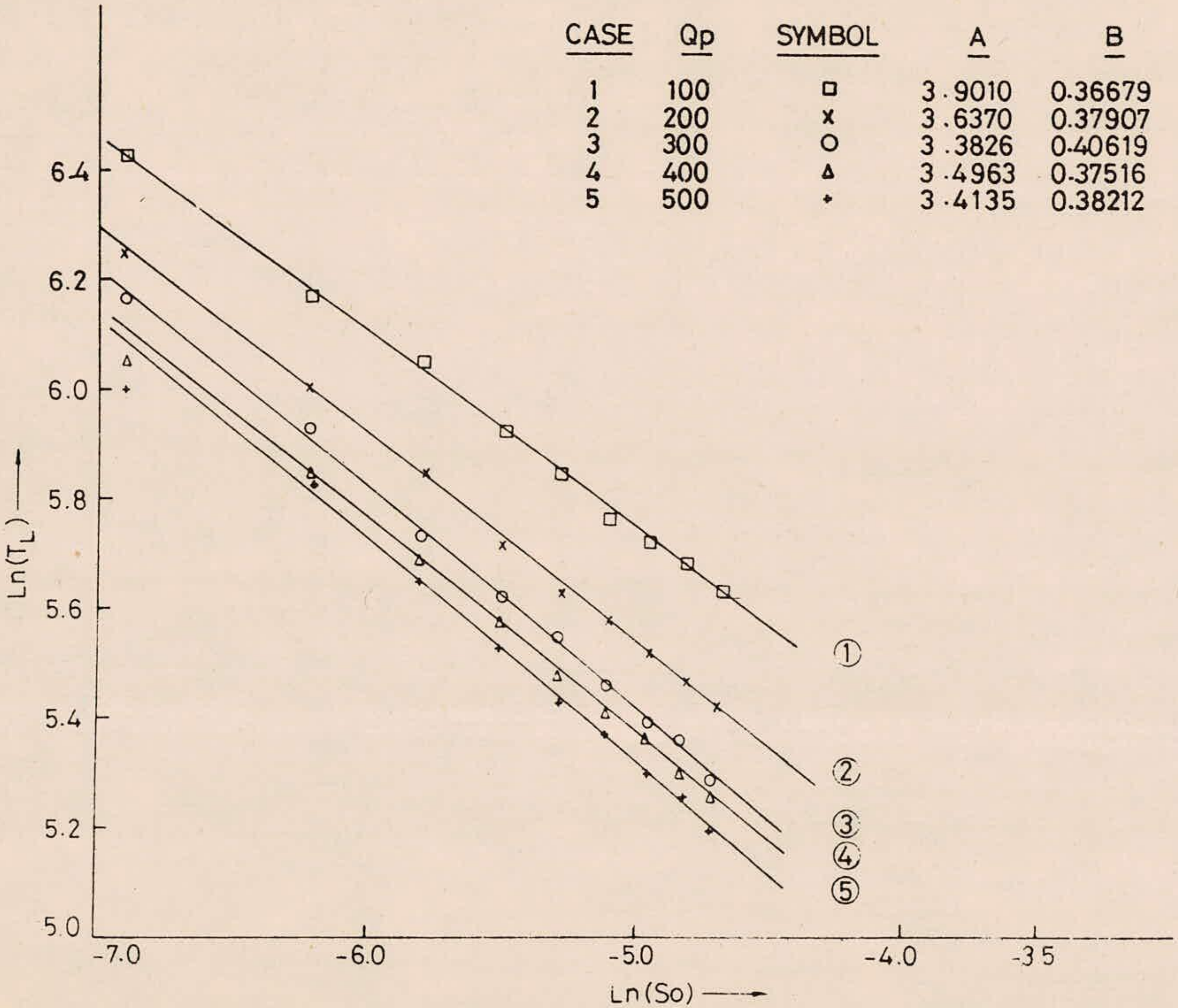


Fig 7 LOG-LOG RELATIONSHIP BETWEEN So AND T_L WITH W = 10'-0" AND r = 1 15

$Ln(T_L) = A - B \ Ln(So)$

CASE	Qp (CUSECS)	SYMBOL	A	B	Cr
1	100	□	3.7295	0.39673	0.9992
2	200	×	3.6194	0.39295	0.9949
3	300	Δ	3.6649	0.37343	0.9861
4	400	O	3.7467	0.35252	0.9742
5	500	+	3.8146	0.33469	0.9690

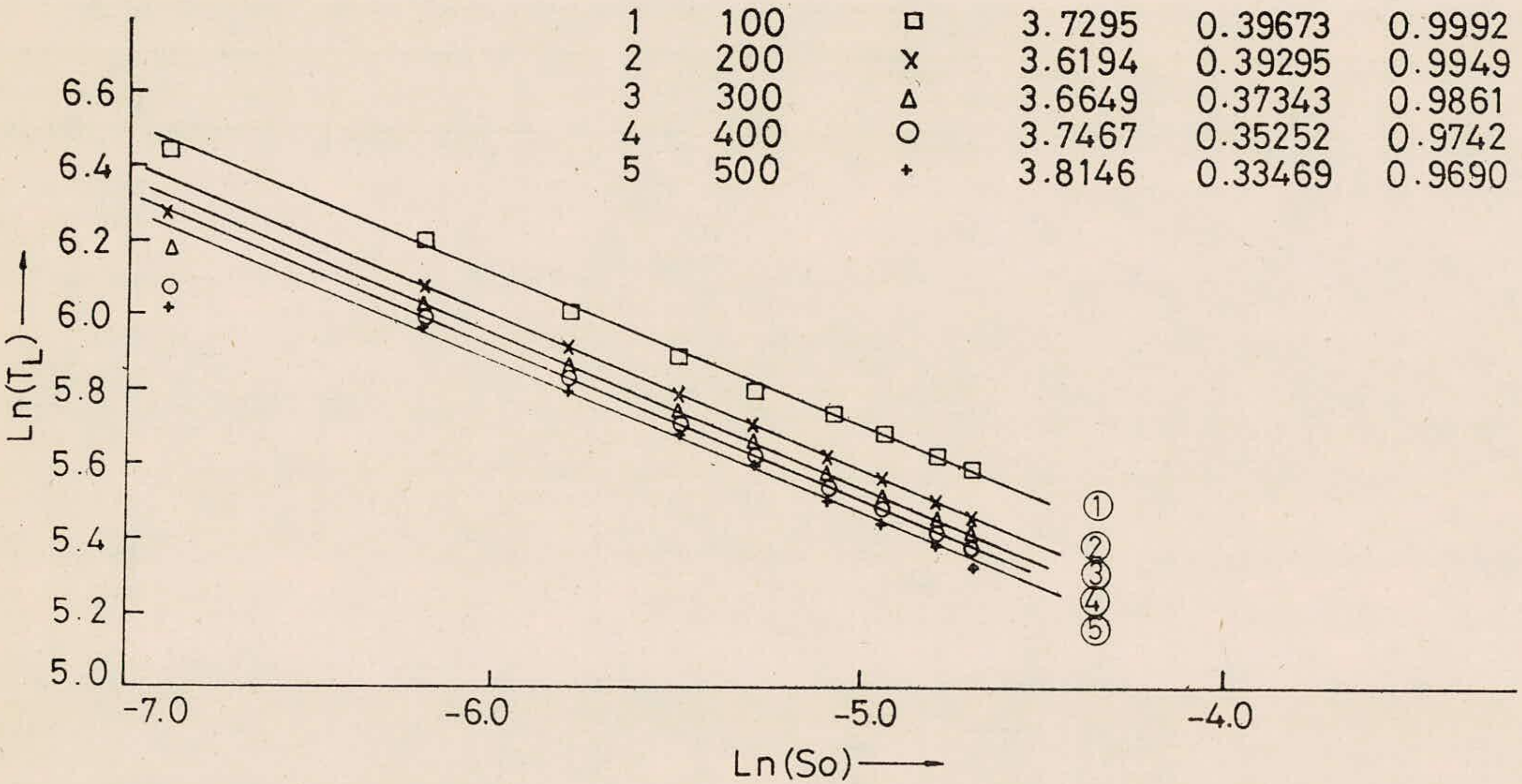


Fig.8. LOG-LOG RELATIONSHIP BETWEEN So AND T_L WITH W = 6'-0" AND r = 1.15

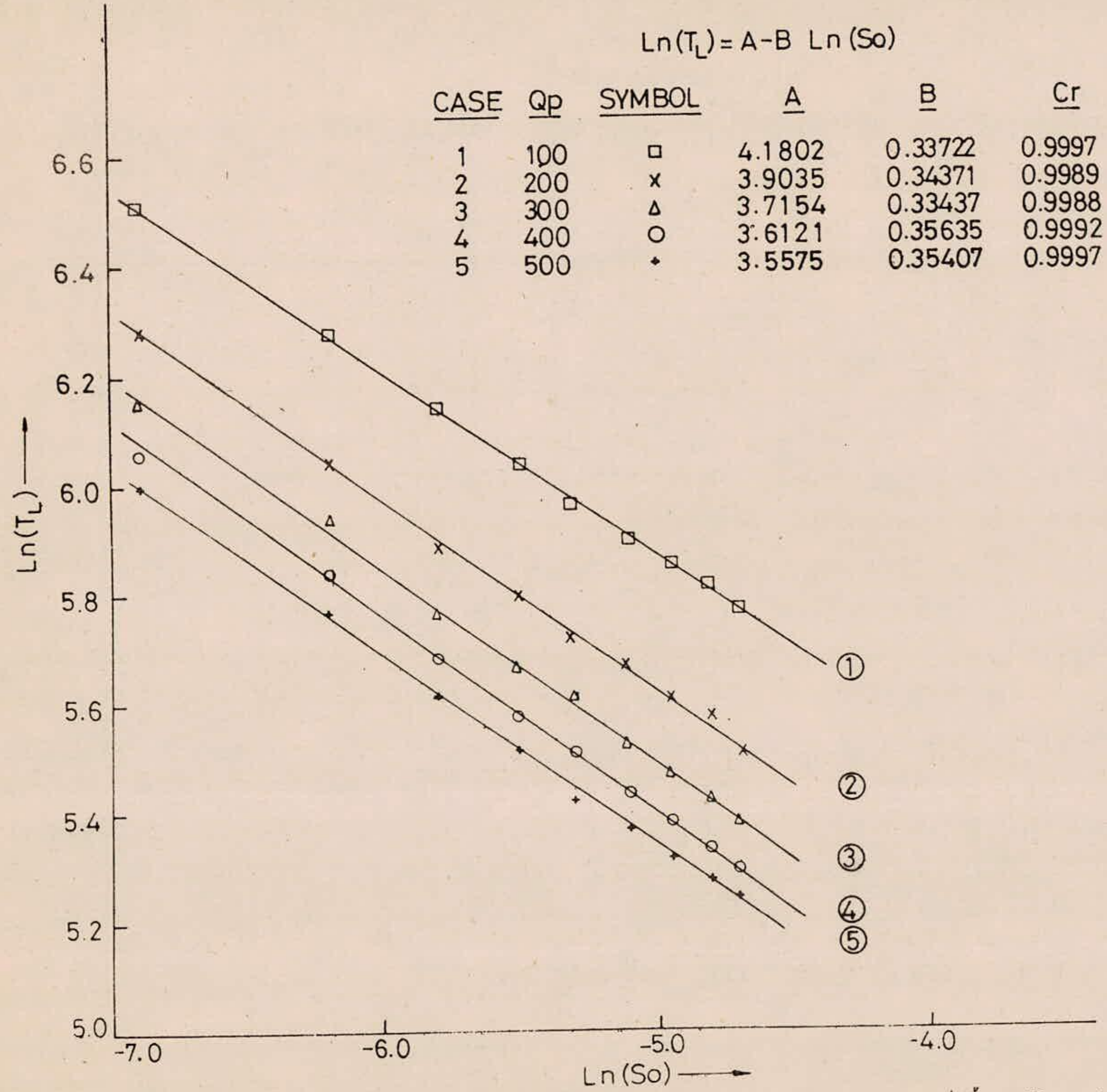


Fig.9. LOG-LOG RELATIONSHIP BETWEEN So AND T_L WITH W = 18'0" AND r = 1.15

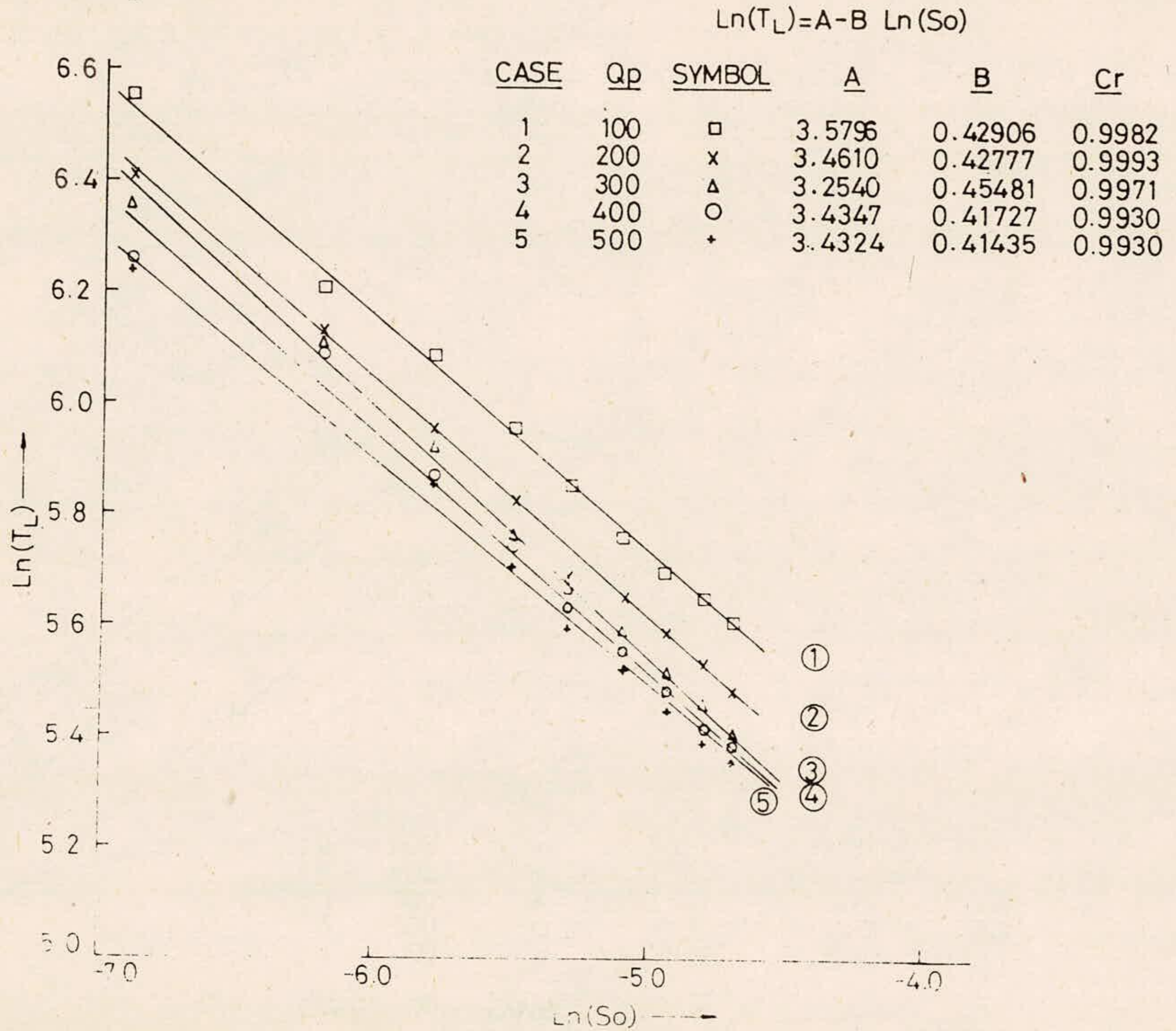


FIG. ... LOG-LOG RELATIONSHIP BETWEEN So AND T_L WITH W=6'0" AND

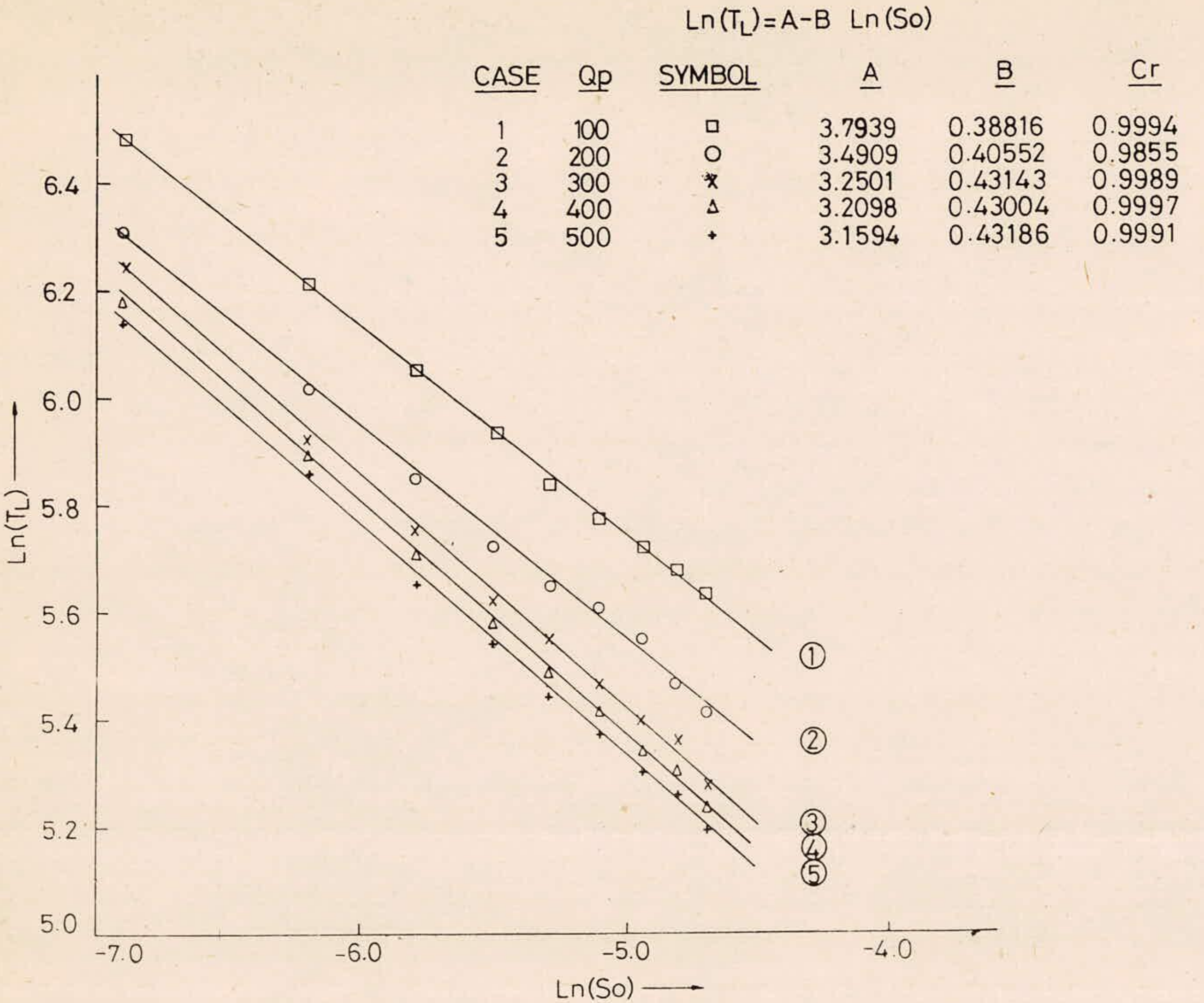
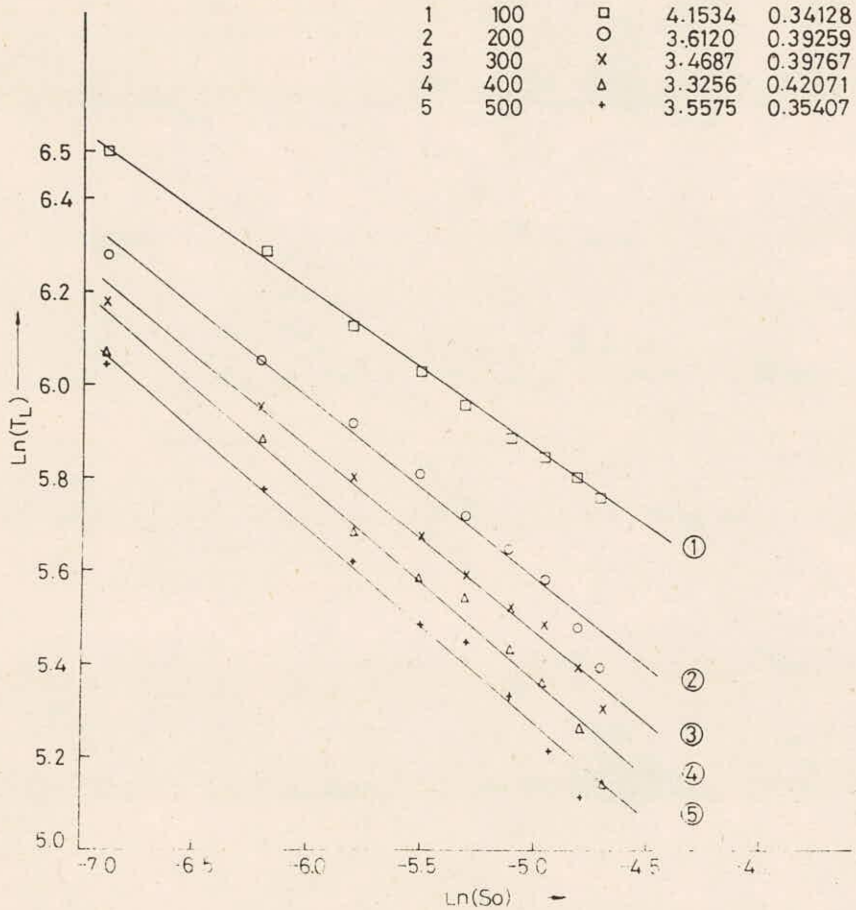


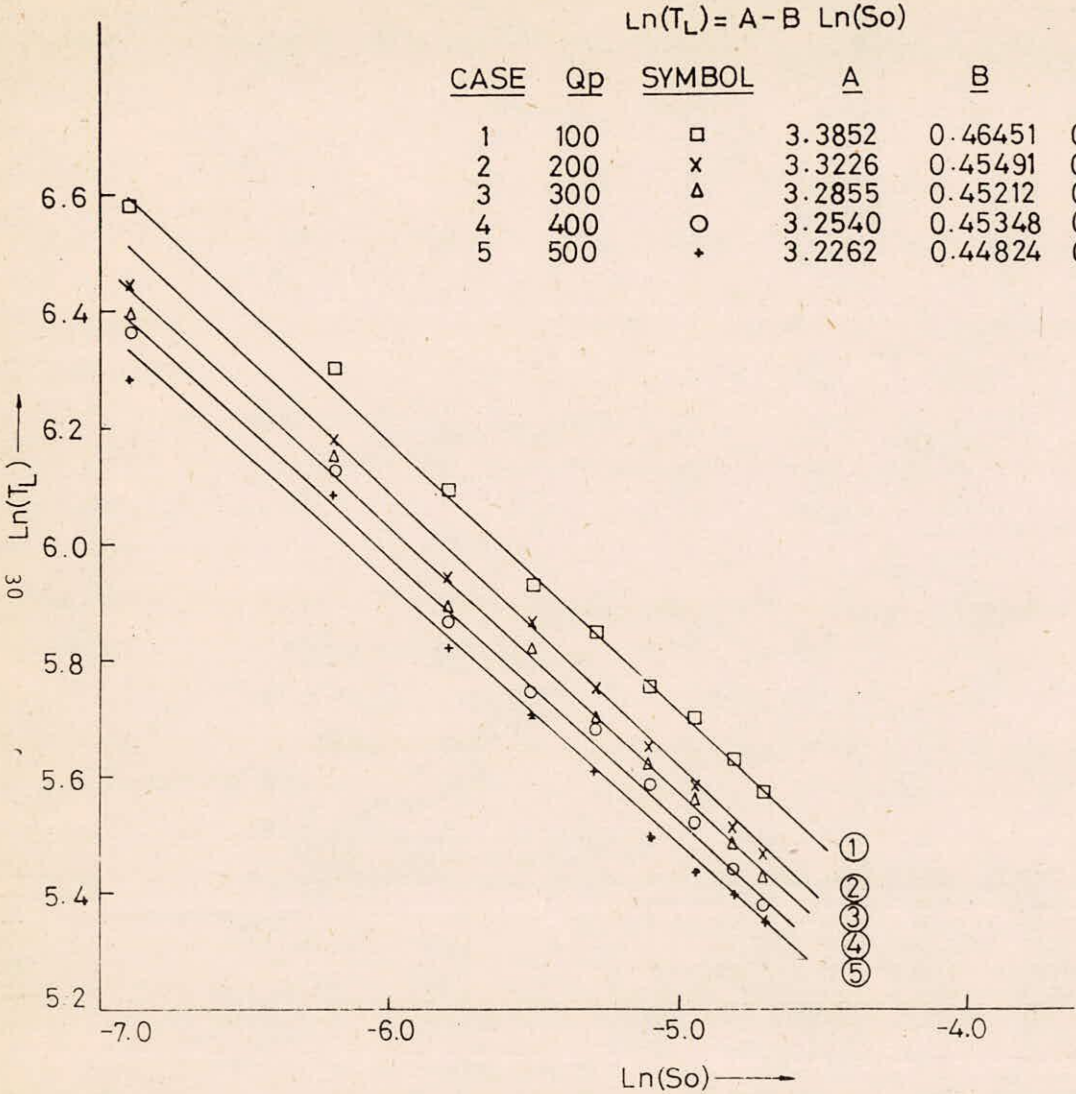
Fig.11. LOG-LOG RELATIONSHIP BETWEEN So AND T_L WITH W = 10'0" AND r = 1.50



$Ln(T_L) = A - B\ Ln(So)$

CASE	Qp	SYMBOL	A	B	Cr
1	100	□	4.1534	0.34128	0.9992
2	200	○	3.6120	0.39259	0.9910
3	300	×	3.4687	0.39767	0.9699
4	400	△	3.3256	0.42071	0.9799
5	500	+	3.5575	0.35407	0.9997

Fig 12 LOG-LOG RELATIONSHIP BETWEEN So AND T_L WITH [illegible]



$$Ln(T_L) = A - B\ Ln(So)$$

CASE	Qp	SYMBOL	A	B	Cr
1	100	□	3.3852	0.46451	0.9990
2	200	x	3.3226	0.45491	0.9982
3	300	∆	3.2855	0.45212	0.9958
4	400	o	3.2540	0.45348	0.9977
5	500	+	3.2262	0.44824	0.9947

Fig.13 LOG-LOG RELATIONSHIP BETWEEN So AND T_L WITH W = 6'0" AND r = 1.75

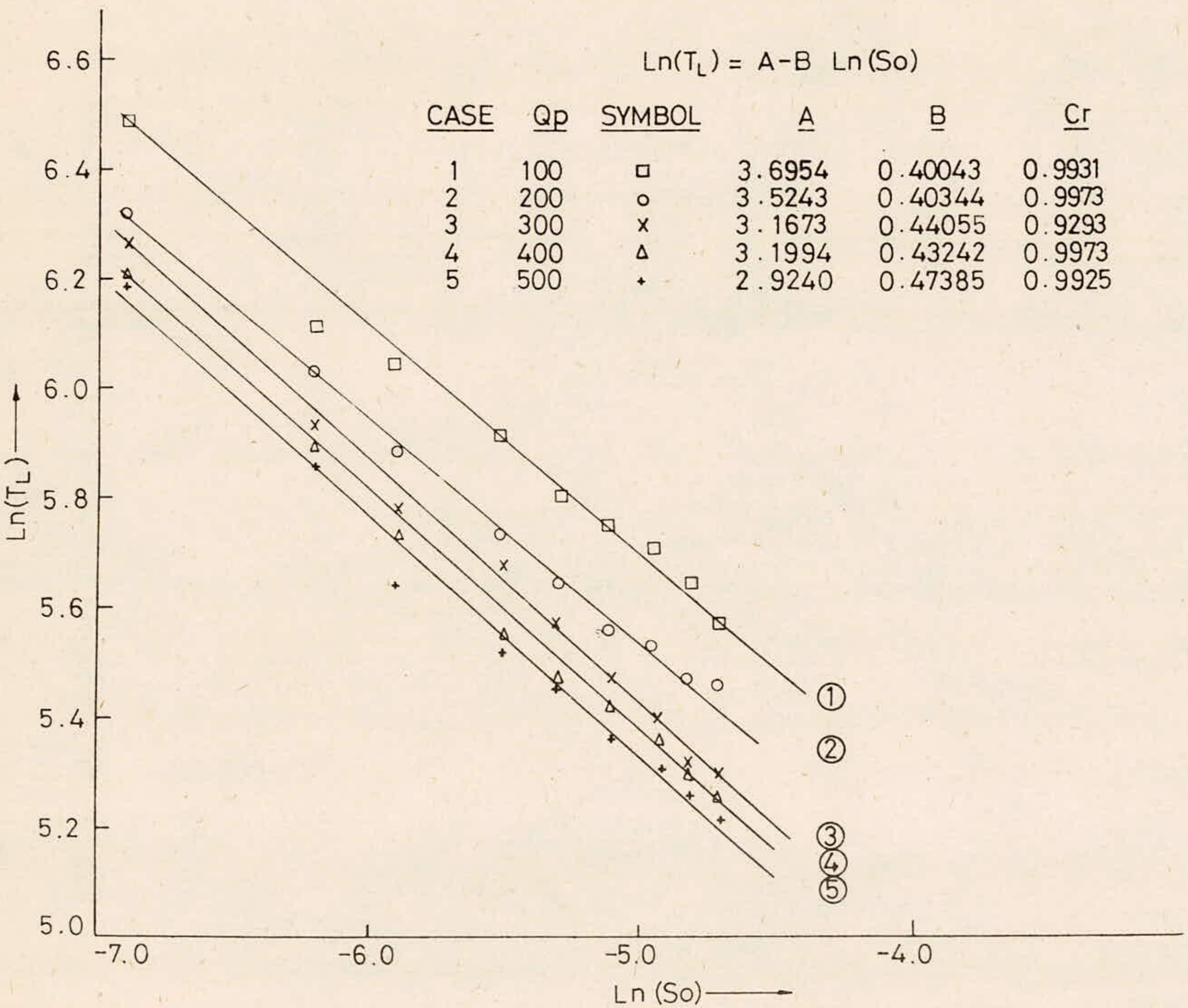


Fig.14 LOG-LOG RELATIONSHIP BETWEEN So AND T WITH W=10'-0" AND r=1.75

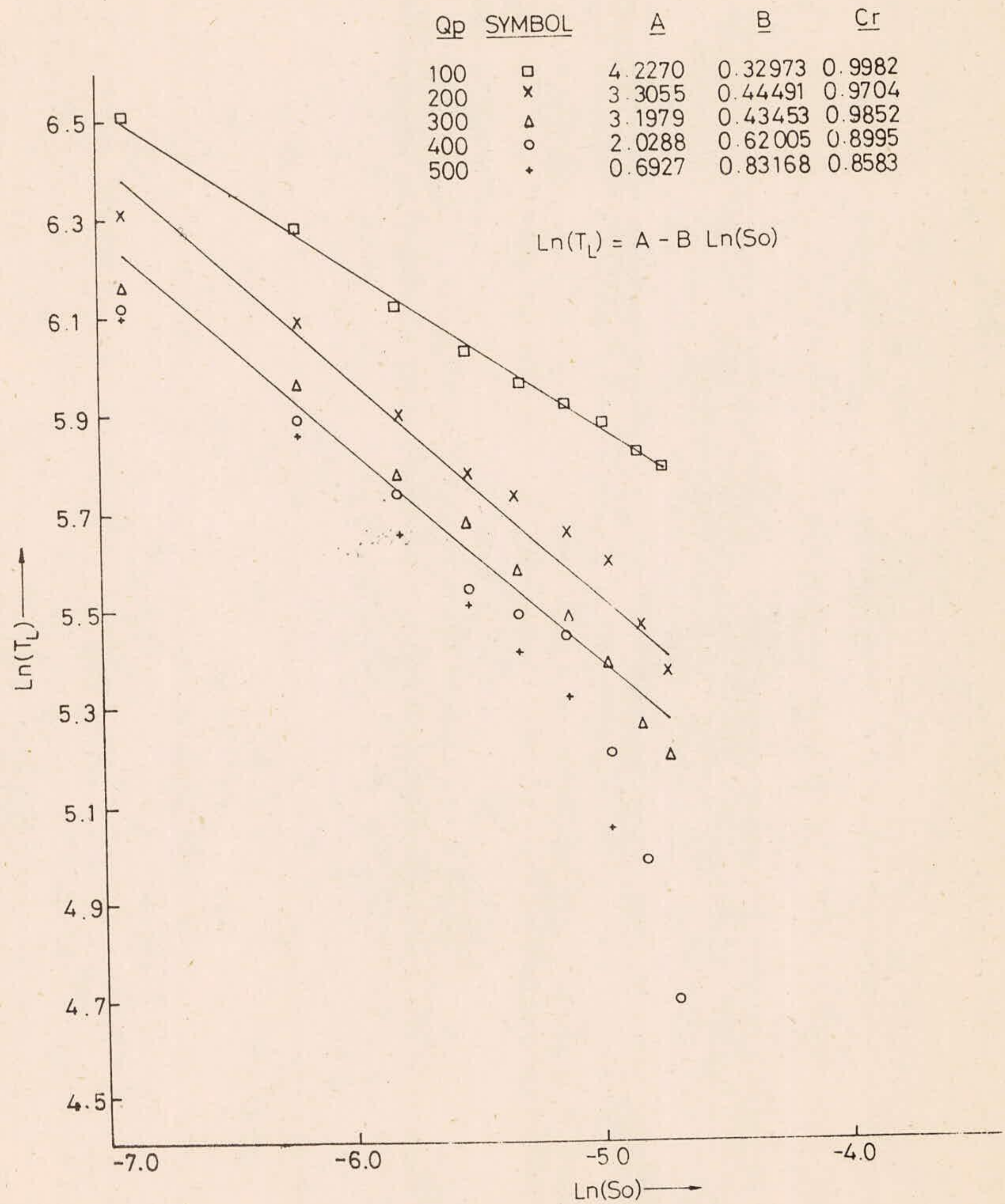


Fig.15. LOG-LOG RELATIONSHIP BETWEEN So AND T_L WITH W=18'-0" r=1.75

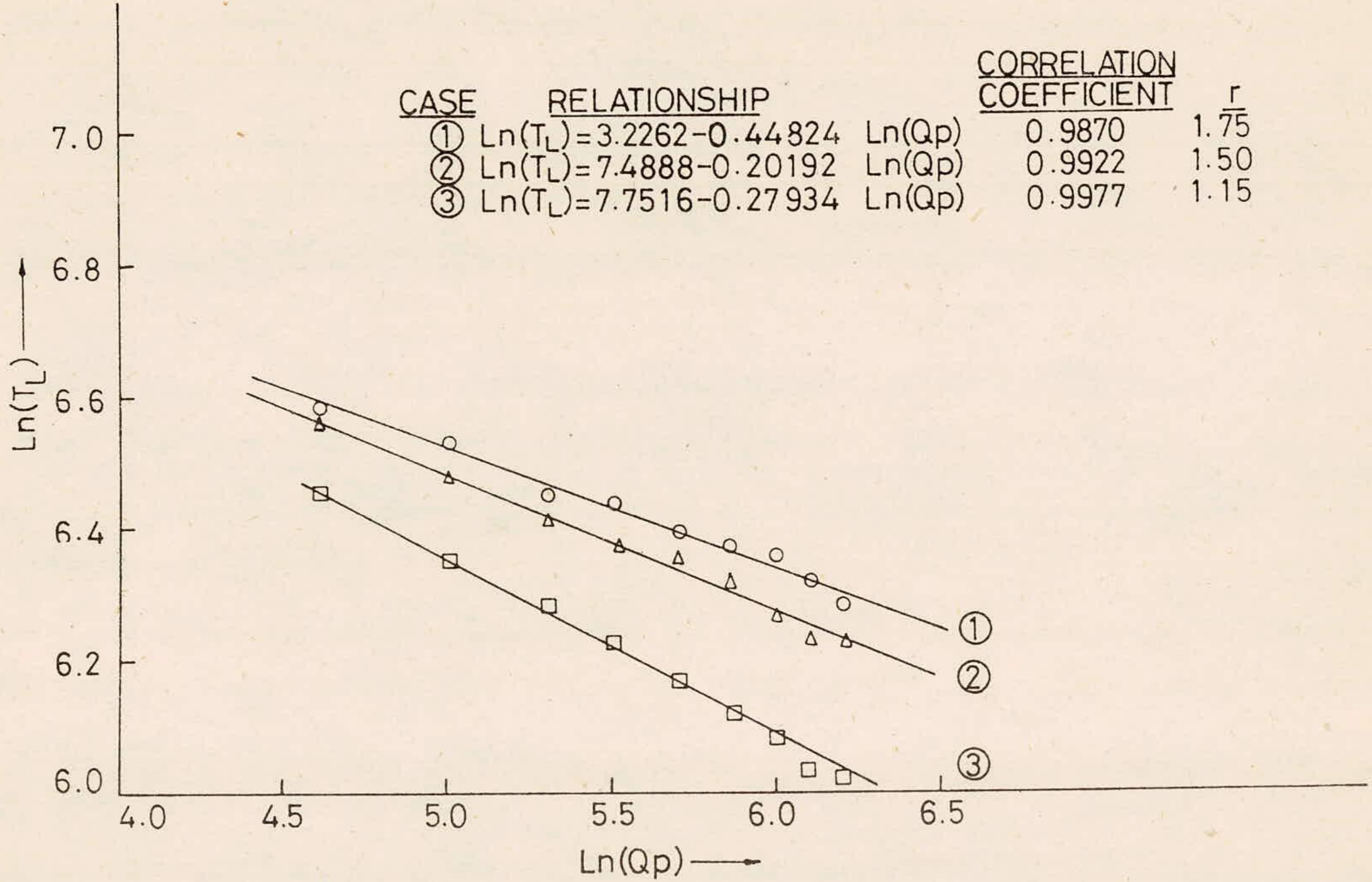


Fig.16. LOG-LOG RELATIONSHIP BETWEEN Qp AND T_L FOR W=6'-0" So=0,001

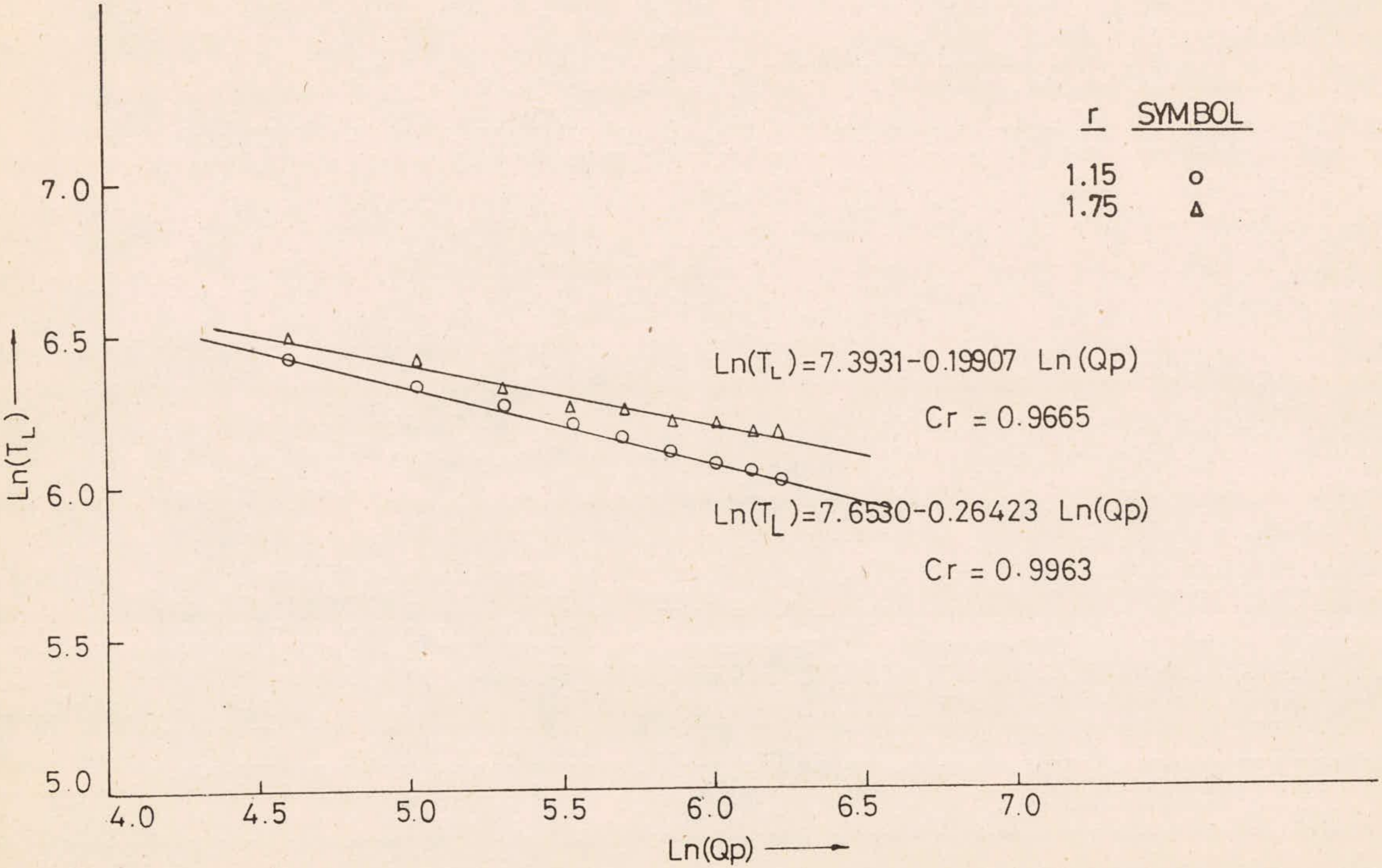


Fig. 17. RELATIONSHIP BETWEEN Qp AND T_L FOR So = 0.001 & W = 10'-0"

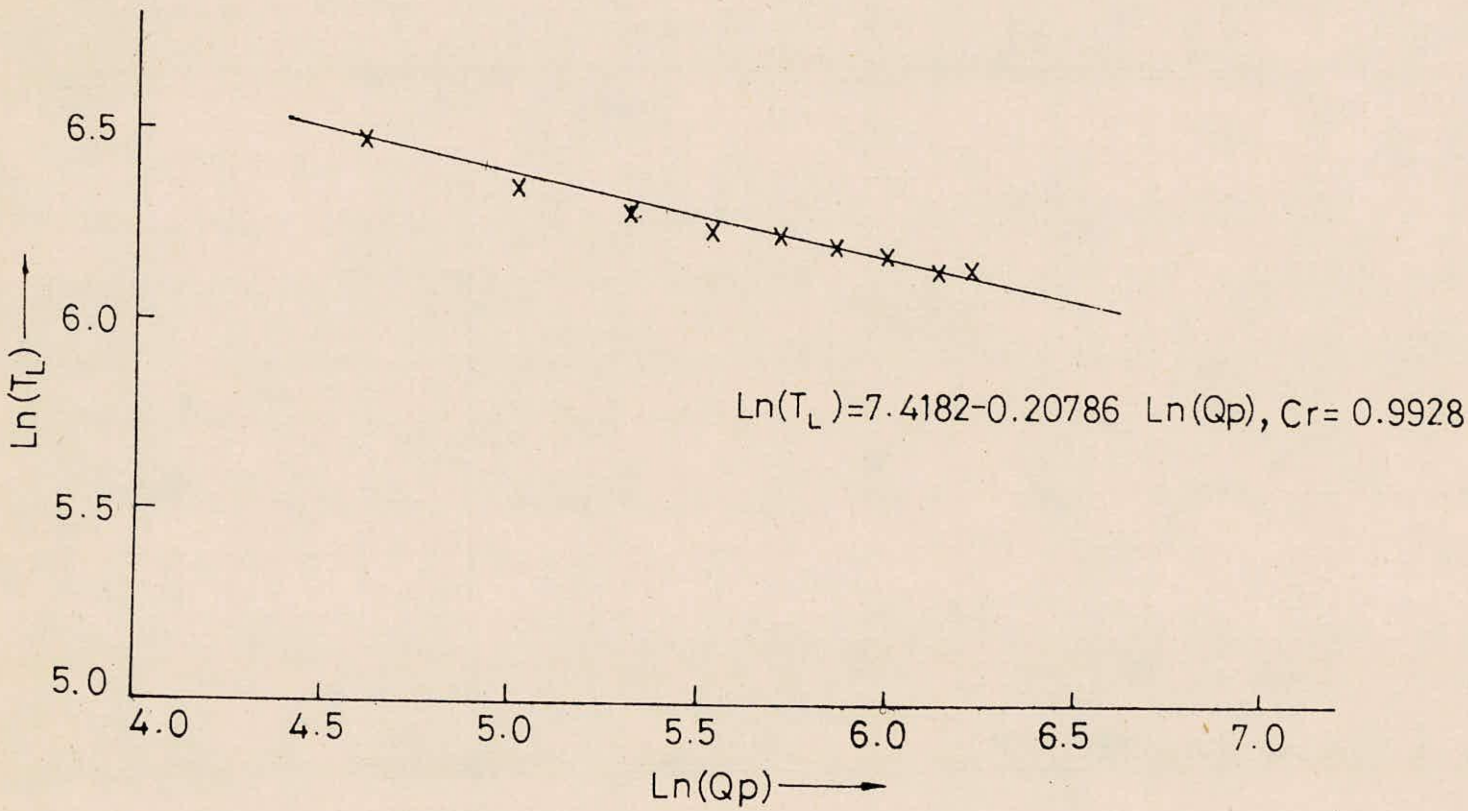


Fig. 18. LOG-LOG RELATIONSHIP BETWEEN Qp AND T_L FOR W=10'-0", So=0.001 & r=1.50

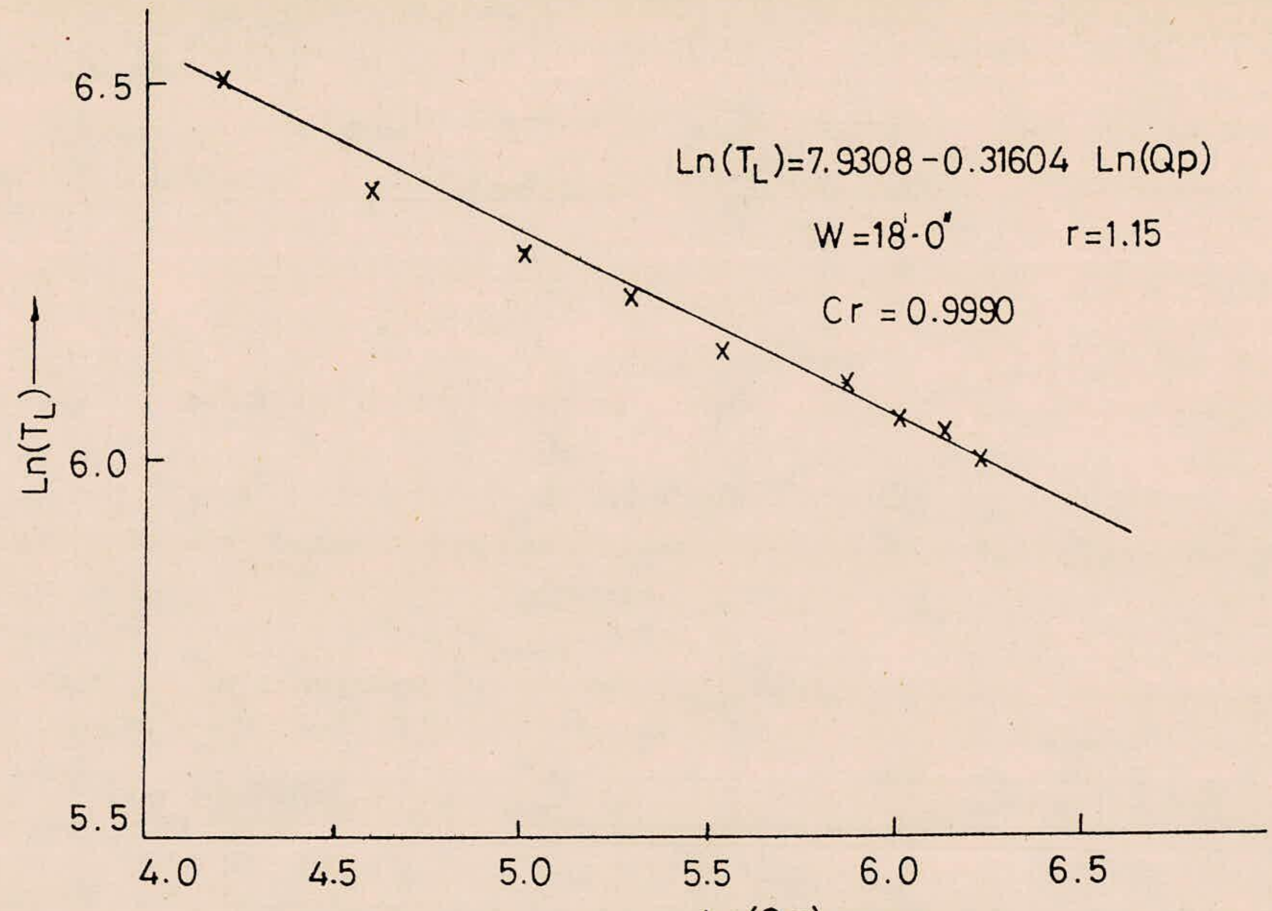


Fig.19. LOG-LOG RELATIONSHIP BETWEEN Qp AND T_L

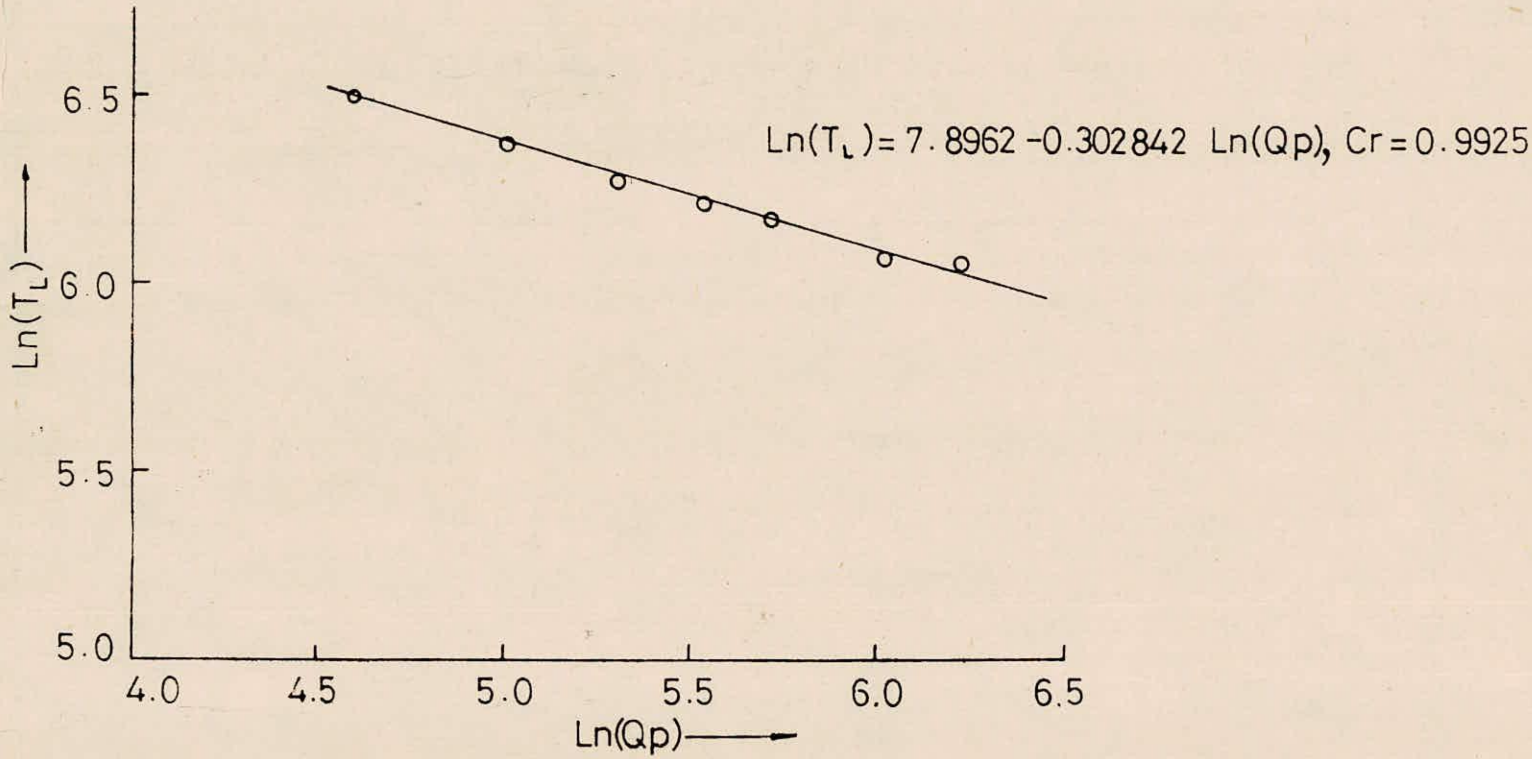


Fig.20. LOG-LOG RELATIONSHIP BETWEEN Qp AND T_L WITH W=18'·0" So = 0.001 AND r = 1.50

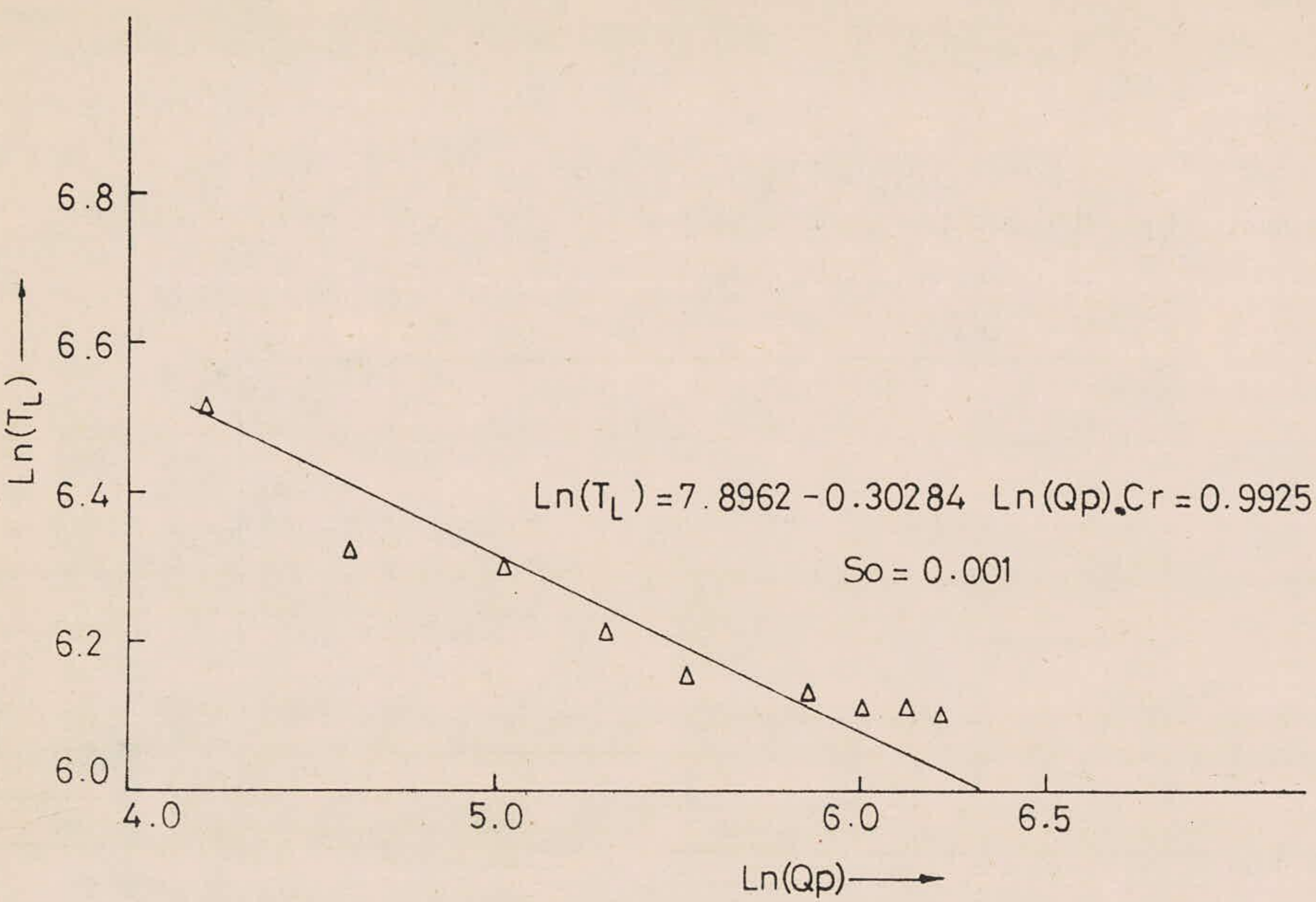


Fig. 21. LOG-LOG RELATIONSHIP BETWEEN Qp AND T_L WITH W=18·0 AND r =1.75

6.0 DISCUSSION OF RESULTS

6.1 Dimensionless Hydrographs

Two distinct dimensionless hydrographs were arrived for each shape factor of the inflow hydrograph; one corresponding to a given bed slope with varying widths of the channels the other corresponding to a given width of the channel with varying bed slopes. The dimensionless hydrographs corresponding to small shape factor of the inflow hydrograph have sharp and distinct peak than those hydrographs corresponding to higher shape factors. This may be attributed to the reason that the inflow hydrograph with smaller shape factor has sharp and distinct peak discharge than that inflow hydrograph with higher shape factor. It can also be inferred that for a given bed slope and shape factor, the non-dimensional hydrograph is not affected by various widths of channels.

Similarly for a given width and shape factor, the non-dimensional hydrograph is not affected by different bed slopes of the channel. However the existence of different non-dimensional hydrograph for different shapes of inflow hydrograph reveal that the routed hydrograph is affected by the shape factor.

6.2 Relationship between Bed Slope and Time Lag of Routed Hydrograph Peak

The relationship between bed slope of channel and time lag in peak flow of routed hydrograph is found to be linear in the log domain corresponding to each peak discharge studied for each hydrograph routed in channels with different widths. The typical relationships arrived are given in Table 1. It can be seen that the correlation coefficient for all the relationship are >0.97.The graphs depicting these relationships are shown in figures (7)-(15). It can be inferred from figure (7) that corresponding to S_o = 0.001 for widths 6 ft and 10 ft for the inflow hydrograph with shape factor 1.15 the time lag T_L values are not following the trend of the rest of the data points. This may be due to some attenuation effects of the flood wave hydrographs due to narrow channel width and steep rising hydrograph. For the same shape factor of inflow hydrograph and for the width of the channel 18 ft, the data corresponding to 0.001 follow the rest of the data indicating that attenuation effect is not at all present when the width of the channel increases. While rest of the relationships between T_L and S_o are perfectly linear in the log domain, the relationship corresponding to channel width of 18 ft. and the inflow hydrograph shape factor = 1.75, is not linear except for peak discharge Q_p = 100 ft^3/sec. The reason for deviating trend from the linear relationship at higher slopes may be attributed to the too much broadening of the hydrograph leading to uncertainty in the determination of exact time of peak flood at the downstream location. Therefore when the width and discharge increases the routing of broad peaked hydrograph further broadens the routed hydrograph leading to uncertainty of

TABLE - 1

Relationship between Bed slope and Time lag of Routed hydrograph peak:

General Relationship $L_n (T_L) = A - B\ L_n(S_o)$

Shape factor (r) 1	Peakflow (Cusec) (Q_p) 2	Sewer width (ft) (W) 3	Coefficient (A) 4	Coefficient (B) 5	Correlation coefficient (C_r) 6
1.15	100	6	3.7295	0.39673	0.9992
" "	200	"	3.6194	0.39295	0.9949
" "	300	"	3.6649	0.37343	0.9861
" "	400	"	3.7467	0.35252	0.9742
" "	500	"	3.8146	0.33469	0.9690
" "	100	10	3.9010	0.36679	0.9993
" "	200	"	3.6370	0.37907	0.9995
" "	300	"	3.3826	0.40619	0.9987
" "	400	" "	3.4963	0.37516	0.9984
" "	500	"	3.4135	0.38212	0.9937
" "	100	18	4.1802	0.33722	0.9997
" "	200	"	3.9035	0.34371	0.9989
" "	300	"	3.7154	0.33437	0.9988
" "	400	"	3.6121	0.35635	0.9992
" "	500	"	3.5575	0.35407	0.9997
1.50	100	6	3.5796	0.42906	0.9982
" "	200	"	3.4610	0.42777	0.9993
" "	300	"	3.2540	0.45481	0.9971
" "	400	"	3.4317	0.41727	0.9930
" "	500	"	3.4324	0.41435	0.9930
" "	100	10	3.7939	0.38816	0.9994
" "	200	" "	3.4909	0.40552	0.9855
" "	300	"	3.2501	0.43143	0.9989
" "	400	"	3.2098	0.43004	0.9997
" "	500	"	3.1590	0.43186	0.9991
" "	100	18	4.1534	0.34128	0.9992
" "	200	"	3.6120	0.39259	0.9910
" "	300	"	3.4687	0.39767	0.9899
" "	400	"	3.2256	0.42071	0.9799
" "	500	"	3.5575	0.35407	0.9997

1	2	3	4	5	6
1.75	100	6	3.3852	0.46451	0.9990
	200	"	3.3226	0.45491	0.9982
	300	"	3.2855	0.45212	0.9958
	400	"	3.2540	0.45348	0.9997
	500	"	3.2262	0.44824	0.9947
	100	10	3.6954	0.40043	0.9931
	200	"	3.5243	0.40344	0.9973
	300	"	3.1673	0.44035	0.9298
	400	"	3.1994	0.43242	0.9972
	500	"	2.9240	0.47385	0.9925
	100	18	4.2270	0.32973	0.9982
	200	"	3.3055	0.44491	0.9704
	300	"	3.1979	0.43953	0.9852
	400	"	2.0288	0.62005	0.8995
	500	"	0.6927	0.83168	0.8583

the exact peak arrival time at the downstream section of the channel.

6.3 Relationship between Peak Flow and Lag Time

The typical relationships between Q_p and T_L corresponding to the bed slope of 0.001 are plotted in figures (16)-(21)

The relationship is linear in the log domain with the least correlation coefficient of 0.9665. Therefore one can infer that the relationship between Q_p and T_L is highly linear.

6.5 Relationship between Time Lag of peakflow of the routed hydrograph, Bed slope, width of the channel and peak flow

The relationship between time lag, bed slope, width of the channel and peak flow in the linear form estimated in the log domain were as follows:

$$\ln(T_{L1}) = 5.13201 - 0.367 \ln (S_o) - 0.030 \ln(W) - 0.246 \ln (Q_p) \quad \ldots(18)$$

$$\ln(T_{L2}) = 4.98620 - 0.410 \ln (S_o) - 0.077 \ln (W) - 0.241 \ln (Q_p) \quad \ldots(19)$$

$$\ln(T_{L3}) = 4.98113 - 0.464 \ln(S_o) - 0.142 \ln(W) - 0.269 \ln (Q_p) \quad \ldots(20)$$

Where T_{L1}, T_{L2}, and T_{L3} are time lags of routed hydrograph peak flow corresponding to shapes $\gamma = 1.15$, 1.50 and 1.75 of the inflow hydrograph. These respective relationships have the correlation coefficients 0.9788, 0.9773 and 0.9447 respectively. It can be inferred that the lag time is inversely proportional to bed slope and peak discharge and also inversely proportional to sewer width w.

6.6 Verification of the Dimensionless Hydrograph Procedure:

The relationships described in this study enables one to determine the routed hydrograph for any of the studied inflow hydrograph herein and for any width of the channel and any bed slope between 0.001 and 0.0090 knowing the bed slope, width of the channel and peak discharge. The time lag of peak discharge at the end of 4000 ft. can be estimated using the relationships given by equations (18), (19) and (20). These relationships are valid only for the inflow hydrograph shapes studied herein.

To verify this developed procedure outflow hydrograph at 4000 ft. downstream of the inflow section of the channel having width (W) 10 ft and bed slope 0.0025 was estimated using the above relation for a peak discharge of 175 cusecs. A separate analysis using finite difference solution of St.Venant's equations was carried out for the same parameters as mentioned above. The outflow hydrograph obtained from this procedure and from the procedure based on the derive relationship were compared as shown in figure (22). The good comparison of both these hydrographs verifies the suitability of suggested procedure in this study

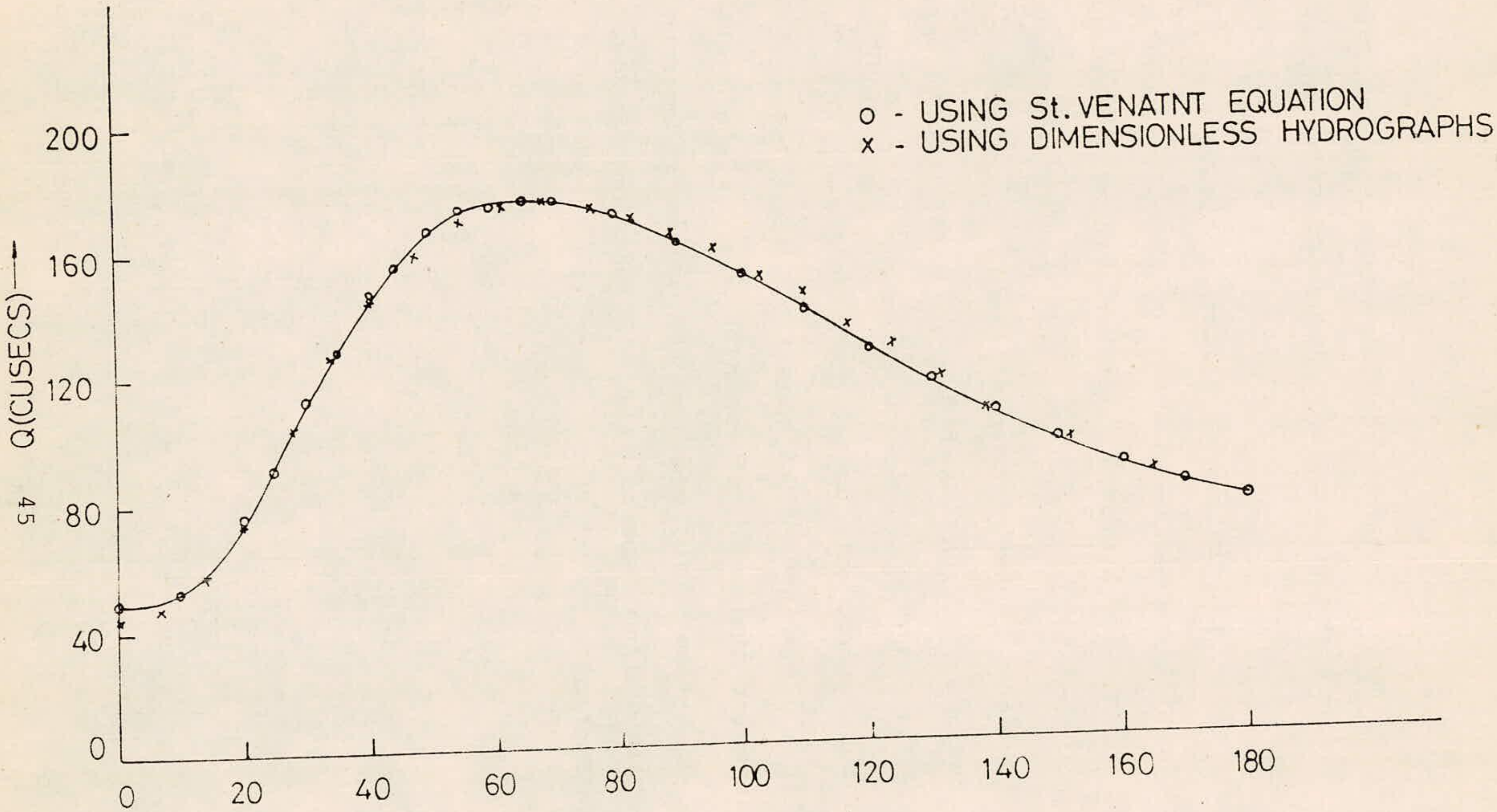


Fig 22 COMPUTED HYDROGRAPH AT 4,000'·0" DOWNSTREAM WITH So = 0.0C25, W= 10 0"

for future use without the need for accessing a computing machine.

6.7 Relationship Between Channel bed slope and Time Lag in Peak for a peaked Hydrograph of Triangular Shape.

In order to study the effect of peakedness of hydrograph on flood wave characteristics in rectangular channel, a peaked hydrograph of triangular shape as shown in figure 23 is used. The corresponding relationship established between channel bed slope and time lag to the peak of outflow hydrographs has been shown in figure 24. The relationship is found to be linear in logarithmic form, with correlation coefficient 0.9993. Therefore, it could be stated that flood waves generated with triangular shapes of inflow also exhibit the characteristics similar to those for the inflow hydrograph shape obtained from Pearson type-III function, as discussed in Section 6.2.

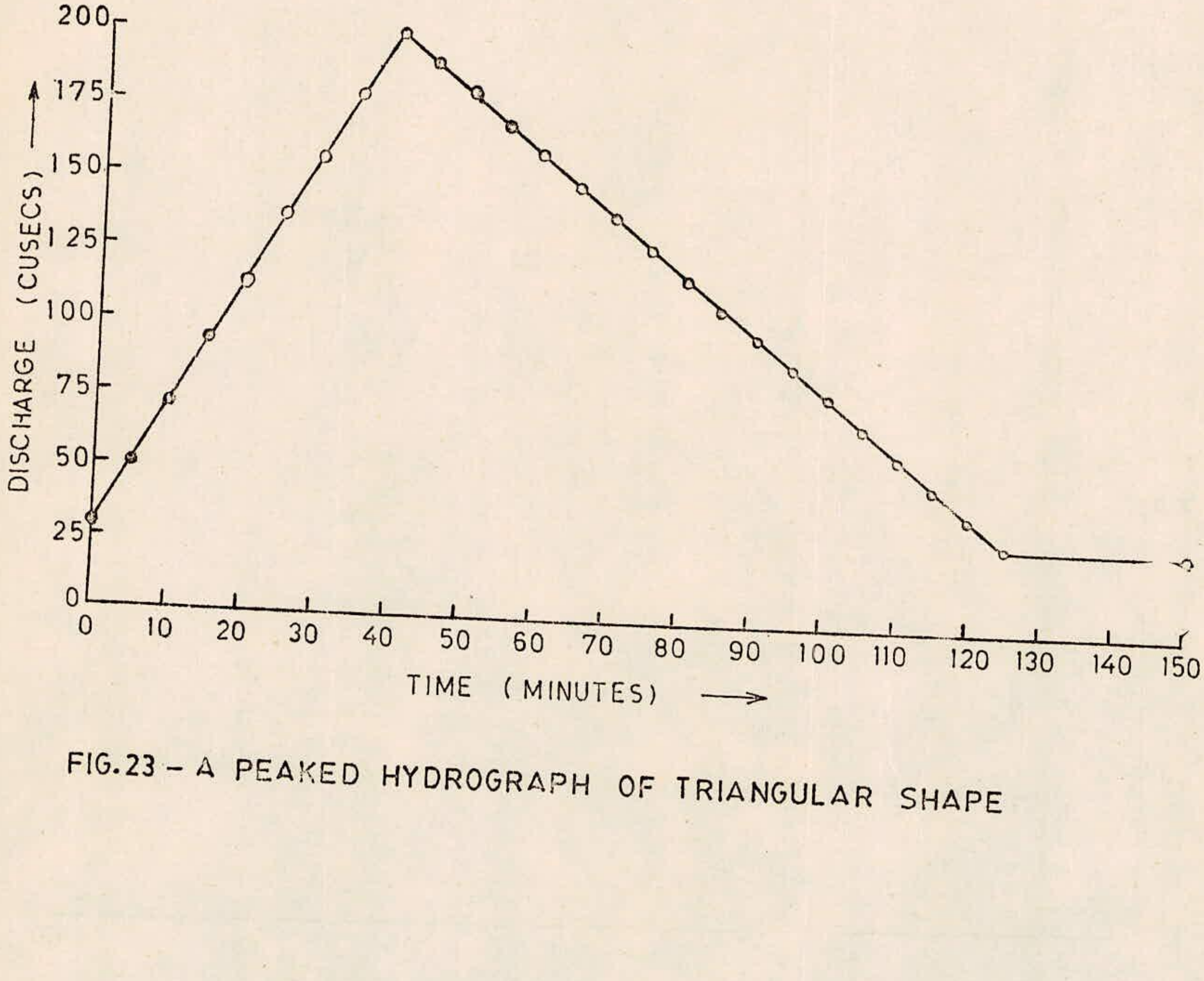


FIG.23 - A PEAKED HYDROGRAPH OF TRIANGULAR SHAPE

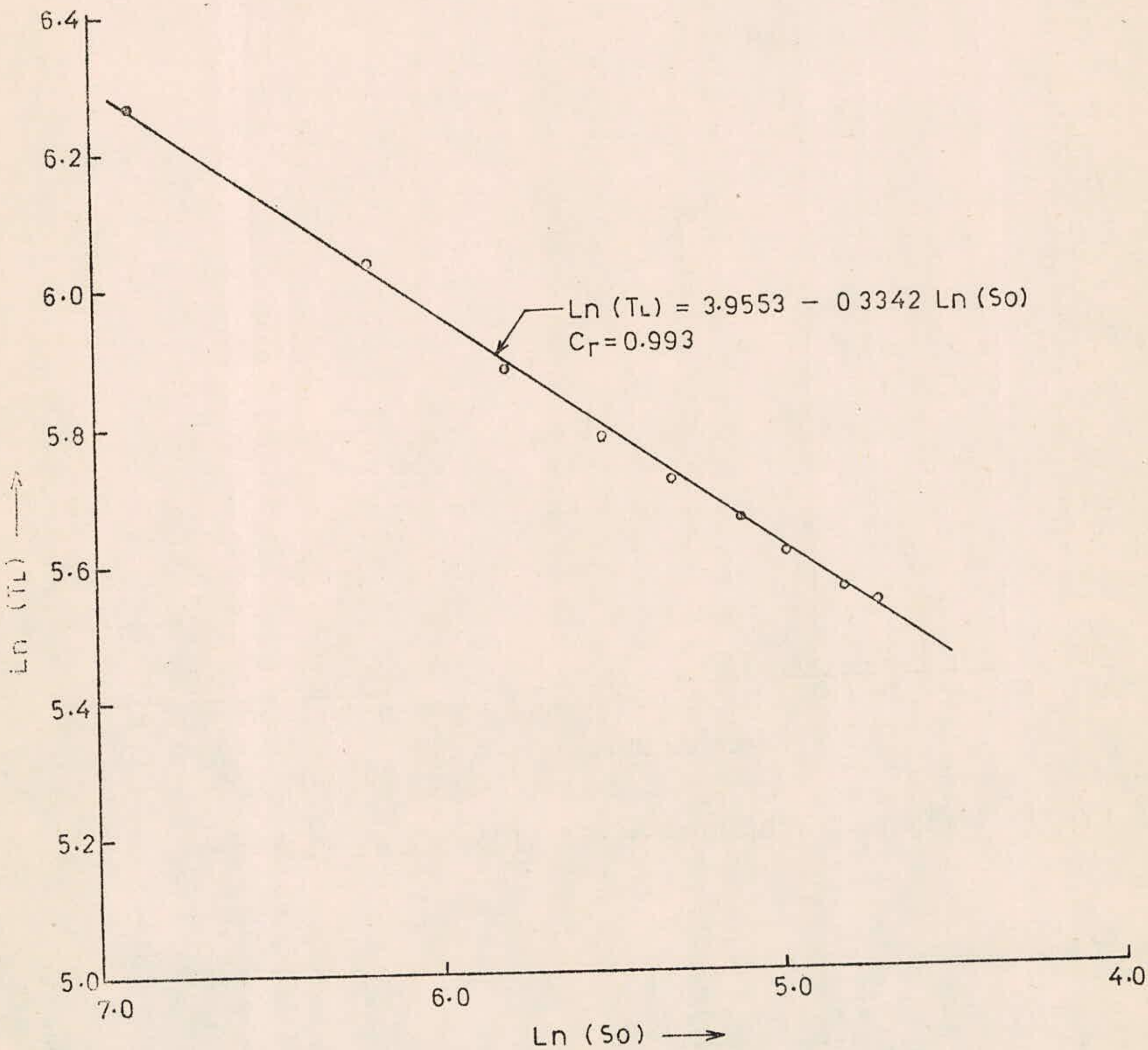


FIG. 24-RELATIONSHIP BETWEEN SO AND T_L WITH W 10.0′ PEAKED HYDROGRAPH WITH TRIANGULAR SHAPE

7.0 CONCLUSIONS AND RECOMMENDATIONS

The study is of preliminary nature with the aim of establishing the relationship between time lag of peak flow of the routed hydrograph with those parameters which directly have influenced over it. It was seen the time lag of peak flow of the routed hydrograph is affected by width and bed slope of the channel, peak discharge and shape factor of the inflow hydrograph. Although roughness factor would have definite influence on T_L, it was considered in this study as constant i.e. 0.014 for concrete bed channels. The results obtained in this study are useful for determining quickly lag time of the peakflow at a particular point downstream of the inlet point of the channel. This information is much useful for the economic design of the channel size of storm sewers as indicated earlier.

A detailed study covering various shapes of inflow hydrographs and widths of channel would enable to get a more generalised relationship for T_L. Further one has to assess whether the shape of the hydrograph has impact on T_L or the volume of water routed. The initial flow in the channel may also impact the variation of T_L. Therefore a considerable amount of study is necessary even for rectangular cross-section channel before going for the establishment of generalised relationship for T_L for other cross-section channels.

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