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MODEL - PROTOTYPE COMPARISONS OF HYDRAULIC STRUCTURES

By

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SYNOPSIS

In this paper mention is made of some of the general factors involved in making model-prototype comparisons of hydraulic structures. Reasons for desiring comparisons, the types made, the difficulties encountered, and the measurements and instruments required are briefly discussed. Citing two of the comparisons made by the Bureau of Reclamation as examples, an account is given of a small irrigation structure, and of the pressure and discharge tests on the 102-inch river outlets of the Grand Coulee Dam. Conclusions drawn from these examples, and a bibliography on model-prototype comparisons concludes the paper.

INTRODUCTION

Although great progress has been made during the last decade in the use of hydraulic models to aid in the design of hydraulic structures, there has been but little progress made in comparing the performance of a model and its prototype. It is of considerable interest and importance to make such a comparison in order to determine the degree with which the model may be relied upon for quantitative results even if the laws of similitude have been rigidly followed. In the field of hydraulic structures this is of particular value when applied to discharge coefficients, water surface profiles, erosion of river beds, and pressures on boundary surfaces, among others. Of nearly equal importance is the fact that reliable prototype measurements will furnish accurate design data enabling more economical designs in the future.

The reasons why little progress has been made in comparing the hydraulic performance of a model and its prototype may be explained by a lack of interest, particularly after the prototype is in satisfactory operation; a natural reliance on model theory; and the difficulties of making prototype measurements. Fortunately, considerable interest is now being shown for making these comparisons and it is believed that this symposium will assist materially in not only helping to create more interest but, perhaps, from the exchange of ideas the difficulties involved in making the comparison will be lessened.

Accordingly, a discussion will be made in this paper of some of the recognized difficulties in procuring prototype measurements, together with the measurements and instruments required. Two types of comparisons between model and prototype will follow, one qualitative, the other quantitative. The former is the easiest and the most frequently made. It consists of describing, for example, the observed performance of some structure for several flows, and then comparing these observations with those made of similar flows in the model. It is recognized that this type of comparison is of particular value only to those most familiar with the structure, is not too convincing to others, and leads us no further to actually verifying the quantitative results of the model. Obviously, a quantitative comparison is the most valuable since it will permit not only direct comparison between several quantitative factors of the model and prototype, but it will also begin to establish the degree with which the model can be relied upon. It is not to be construed that the hydraulic model is not reliable to a high degree, because the laws of similitude in themselves are fundamentally sound when used correctly, but it is of importance to obtain concrete evidence and thereby recognize the influence of any digressions made in conducting the model experiments.

FACTORS MAKING COMPARISONS DIFFICULT

Prototype measurements. The difficulty of obtaining accurate prototype data is the principal reason why correlation work has been delayed. Although several methods are available for measuring the flow, the difficulty lies not only in actually being able to procure the measurements, but also in being able to obtain data applicable to the particular feature of the prototype for which a comparison is to be made. In these respects the discharge, velocity, and water surface measurements are the most troublesome.

The discharge passing a structure may usually be measured at a gaging station downstream. During flood flows, however, it is frequently impossible to obtain an accurate measurement because of a change in river section due to bed movement, or because the peak flow may pass before a complete measurement is recorded. Occasionally a flood will occur at night, whereupon an automatic gage, if installed, must be relied upon or the measurement is lost. On the other hand, it is sometimes necessary to wait many years before a structure operates, and even then a sufficient range of head and discharge may not be obtained. Frequently the discharge passing a structure may be derived from the simultaneous operation of a spillway, outlet works, and powerhouse. In such an event it is usually necessary to wait until the flow is derived from only one source, but by that time the range of measurements desired may have passed. Similarly, if a confluence of streams exists between the structure and gaging station, or if heavy rainfall occurs, a discharge measurement adjusted to be applicable to the structure may be of questionable value.

Perhaps the most trouble is encountered when it is desired to obtain velocity and water surface measurements, not only for comparison with the model, but also for procuring fundamental data. If the flow travels at slow velocities and is confined in narrow channels, these measurements may be made with a minimum of difficulty. If the flow travels at high velocities, however, the measurements are extremely troublesome, regardless of the size of channel. For example, how would one measure accurately the velocity and water surface of the flow in the spillway section of the Grand Coulee Dam or Shasta Dam where the velocity is in excess of 100 feet per second, and the water surface is affected by air entrainment? It is impractical to use current meters or pitot tubes to measure the velocity, or point gages to obtain the water surface.

The futility of using pitot tubes was realized in connection with the Grand Coulee Dam, more from the troubles of maintaining the equipment than from the magnitude of the quantities to be measured. During construction two streamlined, bronze piers about 21 inches high were fastened to bronze base plates at the entrance and on the lip of the spillway bucket. Each bronze pier supports five stainless-steel pitot tubes whose static and dynamic openings are connected within the piers to hydrostatic pressure cells similar to those used for measuring seepage pressures in earth dams. Each cell is connected to an instrument panel in a gallery at a higher elevation by copper tubing extending through the dam. To provide check measurements, a bypass is formed at each cell and connected by copper tubing to manometers in the gallery. In spite of all the precautions made, many of the copper tubes leading to the gallery became plugged and all attempts to clear them not only failed, but some of the diaphragms of the pressure cells were ruptured during the attempts to clear the tubing. It is also feared that debris in the tailwater has plugged or coated some of the pitot tube openings and may have damaged the entire assembly from impact, which may be even more severe after the spillway is operated for the first time in June 1942. As a result, it is doubtful that any reliable measurements can be taken with this equipment, even though careful planning had been made and all precautions were taken to assure an efficient arrangement. It was realized that the pitot tube assembly might be inadequate, so the salt-injection method will be attempted making use of electrode plates, which were also installed at intervals down the spillway section of the dam. As yet no suitable methods have been developed to measure the water surface for the high velocity flow occurring in spillways at high dams or in other open channels.

Difficulties are also encountered when measuring velocities in large outlets under high heads because of the tremendous forces involved, which necessitate extremely rigid instruments or else prohibit the use of any instruments whatsoever. Lack of access also eliminates any measurements, particularly if previous preparations have not been made for installing the necessary equipment during construction.

The success with which other measurements can be made in the field, such as pressures on boundary surfaces, vibrations, and air-demand, depends more on the careful planning of the instruments, the ability to foresee their workability, and their installation than upon the difficulties of actually being able to obtain the measurements.

Planning and cost. Since the success of any verification depends upon the flow measurements made on a model and its prototype, it is compulsory to formulate plans for the comparison during the laboratory experiments and during the design of the structure. Such plans should provide for quantitative model data covering several flows, not just the maximum, and for the installation of piezometers, gages, and special instruments in the prototype design as dictated by the model tests. As a result of careful planning, the unfortunate condition of having prototype data in one range and model data in another will be avoided, even though it might be possible to regulate the prototype tests to conform to similar ones of the model - a condition which rarely exists.

The success of any planning, moreover, depends upon the circumstances under which the laboratory tests are made. Usually they are performed under considerable strain because of the limited time available after the preliminary design is submitted for testing and before the final design is prepared for construction. Discovery of faults in design during construction also requires model experiments to be made quickly so as not to interfere with the progress of construction. Because of this constant rush the model data usually do not cover the desired range, thorough checks on the data are not possible, and the adequacy of the prototype measuring equipment may suffer.

If some flow measurements have been made over a period of years at a structure never previously tested by a model, it is of considerable interest to construct a model and secure the data necessary for a comparison. This reversed procedure has been tried at the University of Iowa as an academic study, and was used recently at the Bureau of Reclamation, but for the primary purpose of revising hydraulic conditions at an existing structure.

The cost of taking field measurements will be greatly reduced if plans provide for the installation of equipment during construction.

Once the proper arrangements have been made, the actual cost of making the measurements will be small in comparison to the entire cost of the structure itself. The cost of instrumentation, on the other hand, is relatively high and in many cases prohibits any field measurements.

Properties of the model. Not all of the difficulties in making comparisons are attributed to the prototype tests, in fact, most of them must be traced directly to the model experiments, assuming, of course, that the prototype data are reasonably accurate. This is evident since any comparison actually is a check on the model tests, not on the prototype. In open channel studies a small error is introduced because it is usually impossible to select a scale ratio which will permit rigorous similitude between model and prototype roughness, the model surfaces being too rough. During experiments on different scale models of sluices in the Assuan Dam[1], it was found that models having a surface roughness commonly used today agreed with the prototype discharge within two percent. When the model surfaces were artificially roughened with sand, however, the model discharge decreased five percent. From measurements on five geometrically similar models of spillways, Eisner[2] showed that there was a definite variation in the coefficient of discharge with the scale ratio. The relative roughness of the boundary surfaces was believed to be the important factor, the effects of viscosity being almost negligible. Because of the uniform variation between the coefficients of discharge and the relative roughness, Eisner predicted the prototype coefficients reasonably well, in comparison to extrapolating by the Froude number.

A much greater error is introduced in model studies of closed conduits because it is practically always impossible to operate the model at Reynolds numbers as high as those obtained at the prototype. It is important to realize, therefore, that the model friction losses are too large, and that a correction should be made to the model data before estimating any corresponding prototype values. Other errors

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1"The Similarity of Motion of Water Through Sluices and Through Scale Models: Experiments with Models of Sluices of the Assuan Dam," by H. E. Hurst and D. A. F. Watt, Proceedings of the Institution of Civil Engineers, London, vol. 218, 1923-24, pp. 72-112.

2"Überfallversuche in verschiedener Modellgrösse," by F. Eisner, Zeitschrift Für Angewandte Mathematik und Mechanik, vol. 11, 1931, p. 416.

introduced by such fluid forces as capillarity and elasticity are negligible in the problems usually studied in hydraulic laboratories. In any event it is well to know the requirements for similitude, how to apply them, and their effects on the results.

In regard to the model data, they are usually more accurate than the prototype measurements, since errors in the field are not controlled as easily as they are in the laboratory. In this regard it should be determined if the model and prototype dimensions are geometrically similar, because frequently the field structure may not be constructed in accordance with the dimensions used for the model.

PROTOTYPE FLOW MEASUREMENTS

Since it is necessary to take nearly the same measurements in the field as in the laboratory, instruments should be available for measuring the discharge, pressure on boundary surfaces, water surface elevations and profiles, velocity, vibration, air-demand, bed movement and for photography. Keeping in mind the difficulties mentioned in obtaining prototype measurements, several well-known methods will be outlined. Although the instruments used in the laboratory and in the field are basically similar, the prototype equipment must be designed to withstand tremendous forces and be convenient for use. With little precedent to follow, one must rely on good judgment for the design of the instruments and, at the same time, be able to foresee their efficacy.

Discharge. Open channel flows can usually be measured by current meters, pitot tube or pitometer traverses, weirs, Parshall flumes, floats, and by the salt-injection method. The particular method used depends on the accuracy required, the size and type of channel, quantity of water, and its velocity. Accordingly, a current meter is most frequently used in rivers, lined channels, and in canals. Smaller quantities in canals are measured with weirs or Parshall flumes. For high-velocity flow in lined channels, the salt-injection method is used unless the discharge measurement can be made at some other place where a more convenient method could be used.

Discharges in closed conduits are obtained by pitot tube traverses, the salt-velocity method, the Gibson method, and by venturi meters when installed in the conduit. These methods are adaptable only if plans have been made for their use, since it has been pointed out that it is often impossible to gain access to many conduits in dams. If access is impossible, the discharge probably could be obtained downstream at a gaging station.

In most of these methods a calibration of the instruments is required, and it is debatable whether the calibration of some of them will be the same at the test site as it is at the place of calibration. Using a rating curve at the prototype which has been established from model tests obviously defeats the purpose of making a comparison.

Pressure. Piezometers, which are used almost exclusively to obtain pressures on boundary surfaces, should be installed on the prototype in positions similar to those on the model. The installation of piezometers in two of the 102-inch diameter river outlets of Grand Coulee Dam is offered as an example; results of the tests are discussed later. Piezometers in these outlets have been placed on the vertical and horizontal diameters. Each opening consists of a high-tension bronze plug drilled with a 1/4-inch hole and screwed into the steel plate lining of the conduits, with the orifice end projecting about 1/16 inch past the interior surface. After this projection had been ground flush with the surface lining, a 1/16-inch radius was formed in the orifice end and all burrs were removed. A short length of 1/8-inch standard brass pipe connects each plug to a steel junction box, where a reducing coupling is used to connect each piezometer plug to 3/8-inch outside diameter by 0.035-inch wall, soft copper tubing. Junction boxes placed opposite each plug are connected by 2-inch galvanized conduit, through which the soft copper tubing is collected and lead to six headers recessed in an inspection gallery below the outlets. The headers, one placed above the other, are made of 2-inch, extra strong brass pipe, with 1/8-inch brass air cocks placed in rows along the top of each header to make connection with each piezometer line. Each header is connected to a high-pressure line at one end and to a bleeder pipe at the other end, which, in turn, is connected to a pressure measuring instrument through a bypass.

A similar method could be used for measuring pressures on concrete surfaces, except the piezometers would be attached to a metal plate set in the concrete. In any event the installation must be made during construction.

To measure the pressures, it is common practice to use water or mercury manometers, and dial gages. Manometers are not always satisfactory because of their fragility, the fluctuations of the mercury columns, and because of the magnitude of the pressures frequently encountered. Commercial-type dial gages are usually not precise enough. A fluid pressure scale of the type used for turbine testing proved satisfactory in high head tests at Grand Coulee and Boulder Dams. Pressures up to 300 pounds per square inch can be recorded within one-quarter of one pound per square inch up to pressures of 100 pounds, and within one-fourth of one percent at any pressure above 100 pounds with this instrument, which is compact and of simple construction. When reading piezometer pressures, the pressure is transmitted to a copper tube grid mounted in gimbals in a weighing beam in the base of the instrument. The weighing piston, which is lapped fit in the cylinder, is restrained from being forced upward. The resulting downward reaction on the weighing cylinder is transmitted, therefore, to the lower weighing beam, and measured on an upper scale beam mounted above the base. The weighing piston is rotated by a small electric motor while readings are being made to avoid static friction with the walls of the cylinder.

Pressure cells used in combination with a group of piezometers and an oscillograph may be used, but the cost of such equipment is prohibitive in many cases unless it is available from previous experiments. This method has one advantage in that a continuous record of the pressure fluctuations is recorded on the film or sensitized paper in the oscillograph.

In making any pressure measurements either on the model or prototype, the care with which the piezometer openings are installed cannot be overemphasized. Any burr will seriously affect the reading and excessive surface roughness either upstream or downstream from the piezometer opening will influence the measurements. It is essential to have a high-pressure water line for flushing the piezometer lines clear of air bubbles, and the tubing from the piezometer to the measuring instrument should be alined, if possible, to eliminate any bends or sags that might trap air.

Water surface. While these measurements are not as important as the discharge and pressure, nevertheless a check is desirable on such design assumptions as freeboard and roughness coefficients. In connection with the work of the Society's Special Committee on Hydraulic Research, the problem of air entrainment in high-velocity flow requires, in part, a measurement of the water surface in open channels at field structures. In comparison to the low-velocity flow in models where the water surfaces are relatively smooth and well defined, there is actually no definite water surface for the high-velocity flow at field structures, but only a mass of spray and aerated water. Efforts to point gage such a water surface in a narrow wasteway in the Yakima project, Washington, were in a measure successful, but only an "average" water surface was actually obtained.

Although point gages may be useful for measuring water surfaces in narrow channels, it might be more feasible at wide channels to use a system of cables to manipulate a heavy target, which could be located by transits. A profile of the water surface could also be obtained by suspending a plumb-bob from the training walls, or from a grid painted on the training walls, but such measurements would be affected by the wall friction.

For measuring water surface elevations in a reservoir, use has been made of the familiar staff gage and of several commercial instruments. Those based on the principle of a long air column transmitting the pressure should be avoided because of the effect of temperature change on the air column, and because of possible air leaks in the tubing.

Velocity. The methods available are directly associated with discharge measurements, so use is made of current meters, pitot tubes, salt-injection method, and floats. Although somewhat unsuccessful at

Grand Coulee Dam, the installation of pitot tubes, as discussed above, may prove successful under different circumstances. The accuracy required for model-prototype comparisons eliminates the use of surface and sub-surface floats.

Vibration. It is frequently desirable to measure the vibrations occurring at a prototype structure, not necessarily to compare with the model to establish a relation between the two, but more to ascertain what effect the vibrations may have on the structure itself. The flutter of nappes discharging over the lips of drum gates on the crest of movable weirs or dams is one example of such vibration,[3] while the discharge at large valves and regulating gates is frequently accompanied by vibrations. Accelerometers and pressure cells were used with an oscillograph to measure vibrations due to fluttering nappes at the Black Canyon Dam in Idaho. The instruments were designed on the principle that a change of resistance occurs with a change in pressure on a pile of carbon disks. The accelerometers measured the accelerations in three directions in the dam, while the pressure cells were used to measure the amplitude and frequency of the inducing forces exerted on the dam by the flutter of the nappes spilling over the lip of drum gates. The instruments were calibrated by noting the deflections produced on the moving elements of the oscillograph under various conditions created on a shaking table. Less complicated instruments such as vibrating reeds, which give only the frequency, have been used elsewhere, the amplitude being determined by magnifying the vibration and using an oscillograph.

Air-demand. It is often necessary and desirable to measure the quantity of air required on a prototype to relieve negative pressures under nappes, in conduits downstream from needle valves and regulating gates, and in regulating gates and valves themselves. It is a difficult problem to determine the size of air ducts required on the prototype by model tests, because of the uncertainty of the similitude involved, thus any check on the prototype will aid considerably in establishing the transference relations. To measure air quantities on the

[3] "Über Schwingungen von Wehren," by Prof. R. Seifert, VDI; Zeits. des Vereines Deutscher Ingenieure, vol. 84, no. 7, Feb. 17, 1940, pp. 105-110.

prototype (and in the model) an anemometer, orifice plate, or pitot tube could be installed in the air ducts. Although not entirely suitable for prototype measurements because of the large quantities involved, a rotameter may be used for model tests.

Bed movement. The scour to river channels downstream from dams and spillways is obtained by surveys or by soundings. These measurements, however, can only be used for a qualitative comparison.

Photography. Still and moving pictures of all flow conditions observed both in laboratory and field work are invaluable for purposes of comparison. Frequently a startling similarity is seen between a model and prototype photograph of some flow phenomenon. For still pictures, the photographic equipment should include at least a standard view camera and a miniature camera, both with fast lenses. The moving-picture camera should be equipped for slow-motion. The higher the number of frames per second the better, because many prototype flows cannot be studied unless considerably slowed down in the moving pictures. Telephoto lens attachments are considered essential for both the still and moving-picture cameras.

EXAMPLES OF QUALITATIVE AND QUANTITATIVE COMPARISONS

Qualitative Comparison

Irrigation structures. Such relatively small irrigation structures as diversion dams, check drops, wasteways, and turnouts offer opportunities of making model-prototype comparisons because of their continuous operation at various discharges during an irrigation season. The primary concern at these structures is to prevent scouring of the canal downstream by using a hydraulic jump stilling basin wherever possible. Consequently, it is sufficient to make only a qualitative comparison of any erosion occurring. Most of the small irrigation structures in use today are based on designs developed over a period of years, so that model studies have not been utilized except for revising some existing structure of inadequate design. Such a condition required a laboratory investigation of check drop 4 in the Sunnyside Main Canal, Yakima project, Washington.

The prototype. Because of a gradual growth in irrigation, the Sunnyside Main Canal had to be increased in capacity. To maintain a proper gradient in the canal, 23 check drops were installed at intervals of about one and one-half miles. These drop structures, which were built during the period 1907-1916, were similar in design, but decreasing in size downstream as the discharge in the canal became less due to deliveries. Each structure consists of a rectangular concrete basin divided into bays by concrete piers which are surmounted by steel brackets. By placing flashboards between each bracket, the depth of flow between successive drops can be regulated. Wing walls at each end of the concrete basin extend into the canal banks at right angles. The drop in water surface at these structures varies from 30 inches for the maximum to 7¼ inches for the minimum, with a maximum discharge of 1,300 second-feet at the first drop gradually decreasing to 520 second-feet at the last one.

Not many irrigation seasons had passed before excessive scouring developed in the canal immediately downstream from the drops, particularly at the larger ones. All attempts to check the erosion failed, until finally the width of the canal below the drops was increased in some cases as much as 50 percent. In 1938 the problem of eliminating this scour was assigned to the hydraulic laboratory of the Bureau of Reclamation, Denver, Colorado.

Model tests. Since the drops were similarly designed, check drop 4 was selected for study, and a 1 to 15 scale model was constructed. The initial tests in the laboratory demonstrated that the model would reproduce the flow conditions and scour existing in the field (figure 1). Further analysis disclosed that, because of the small drop in water surface, standing waves developed instead of a more efficient energy dissipator such as the hydraulic jump. As a result, high surface velocities proceeded downstream in the center of the canal, thereby producing large return eddies along the canal banks which eroded them severely.

A satisfactory solution of this problem was not readily apparent, for not until many designs had been tested was a solution found which would completely eliminate the unfavorable conditions. This consisted of adding curved training walls in the entrance to the drop, extending vertical walls and a concrete floor downstream to form a stilling pool, and placing a deflector between these walls directly above the concrete basin to force the flow under the tailwater. These revisions created a more uniform flow distribution through the structure, and provided energy dissipation sufficient to eliminate the high surface velocities which had caused side eddies to scour the canal banks.

Comparison. After the prototype had been revised, observations of its performance showed that it was acting in a manner similar to that predicted by the model tests. Figure 2 shows the revised design of the model and prototype. The collection of debris at the prototype structure is evidence of roller action in the stilling pool. This is indicative of energy dissipation throughout the entire depth, instead of only on the surface as noticed in figure 1. During the laboratory studies a similar collection of debris was noticed, but it was given little consideration since it interfered with photographing the action of the model. In the field, however, it has been of considerable value in clearing the canal of large amounts of debris, conveniently removed at the drop structure. Three miles downstream at drop 6, which has also been revised, there is little trash remaining in the canal.

The results of this comparison are of considerable satisfaction to those familiar with the problem, but as previously mentioned such a correlation is more convincing to those directly concerned. Nevertheless, this study is offered as convincing proof that model tests are an excellent tool for the hydraulic engineer and that they can be relied upon to a high degree.

Quantitative Comparison

The prototype. Sixty 102-inch diameter outlets have been provided in three tiers of 20 each at the now completed Grand Coulee Dam to release a total discharge of 225,000 second-feet (figure 3). Each outlet has a bellmouth entrance followed by a length of straight conduit, which terminates in an elbow and cone at the downstream face of the dam. Closure of each outlet is made by a service gate and an emergency gate, partial gate openings never being used with ring-seal or paradox gates. Except for a part of the lower tier, each outlet has a steel-plate lining.

The three tiers of outlets at elevation 1136.67, elevation 1036.67, and elevation 935.76 are designated as the upper, intermediate, and lower outlets, respectively. Since the outlets occur in pairs in each tier, 15 feet on centers, further designation is given by east (E) and west (W), and by the block number in which they occur. Figure 4, for example, shows upper outlet 63W discharging down the face of the dam.

During the design of the outlets, plans were formulated for installing piezometers in one intermediate outlet, 51E, and one upper outlet, 51E, 100 feet immediately above. Piezometers were not placed in the lower outlets because they were not completely steel-lined and because detailed model studies were not made of them. In locations similar to those in the model, a series of 11 piezometers have been placed in the bellmouth entrances along both the invert and crown (vertical center-line), and on the sides (horizontal center-line). Similarly, the horizontal part of the two conduits contains 12 piezometer rings in the intermediate outlet, and 10 in the upper outlet; both have 10 rings in the elbow and cone. The installation methods have been described above, while the vertical section of an intermediate outlet shown on figure 5 indicates the distribution of piezometers.

Model tests. A resume of the 1 to 17 scale model tests on the outlets of Grand Coulee Dam has been given in another paper.[4] Suffice it to say here, the tests treated the bellmouth entrance, the profile of the outlets, the elbow and cone, and the deflectors placed in the spillway immediately above the exit of the upper and intermediate outlets, which can be seen in the bottom of figure 4 and figure 5. Measurements on the model were concerned primarily with the pressures on the boundary surfaces, measured with water and mercury manometers, and the discharge for several heads, obtained from weir measurements.

Only the model data from the intermediate outlet tests will be used for comparing with the prototype. Some tests were made in the laboratory

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[4] "Distinctive Features of Grand Coulee Outlets," by Jacob E. Warnock, Civil Engineering, December 1940, pp. 779-782.

on the lower outlets, but no piezometers were installed in the prototype, and since the upper outlets are quite similar to those in the intermediate tier, additional tests were not required.

Prototype tests. Twenty-three tests have been made on the intermediate and eight on the upper outlets. Heads above the exit have ranged from 28 feet to 234 feet for the intermediate outlet, and from 33 feet to 133 feet for the upper outlet. In addition to measuring pressures with the fluid pressure scale, the reservoir elevation, water temperature, tailwater elevation, discharge, and the number of outlets open in each tier were recorded. To make comparisons with the model of the intermediate outlet, it was possible to use only those tests in which the adjacent outlet, 51W, and the upper outlet, 51E, were closed. This not only conformed with the laboratory arrangement, but it also eliminated from consideration any secondary effects on the pressures. These effects were discovered first during a separate model investigation of the bellmouth, and later during the prototype tests when it was noticed that pressures in the bellmouth were slightly lowered when the adjacent outlet 51W was operating, while pressures in the elbow and cone were raised when the upper outlet 51E was operating.

Model-prototype pressure characteristics. The first comparison given on figure 5 is intended to show the characteristics of the hydraulic gradient. Because the field tests could not always be accomplished when the reservoir reached a corresponding elevation, the model and prototype heads shown are not comparable. Upon first consideration it would seem desirable for the heads to agree, but even if they did, an agreement between the model and prototype pressures would be lacking. The reason for this is evident from figure 8, where it may be seen that the model tests were made at much lower Reynolds numbers, hence the friction losses were proportionately greater. Consequently, when the model pressure heads are scaled to the prototype by the Froude law, they will be lower than corresponding prototype values. This fact is generally well-known and is only mentioned to show the effect of not satisfying the similitude requirement that the model should be tested at the same Reynolds number as the prototype. This condition is incompatible with hydraulic models based on Froude's law, in addition to the fact that the extremely high velocities required to comply with the Reynolds law are impossible to obtain in many laboratories. In any event the model results are on the side of safety.

Notwithstanding the lack of complete similitude, a study of figure 5 indicates a certain similarity between the model and prototype gradients. Whereas the prototype data are quite consistent, the model data appear to be erratic at some of the piezometers. More will be said about this in the following discussion.

Pressure drop in bellmouth. To make a more valid comparison than the one in figure 5, the pressure drop in the bellmouth has been analyzed. By choosing a short length of conduit, the effect of the differences in friction is negligible, and by using dimensionless parameters, comparable heads are not required. Accordingly, as shown on figure 6, curves have been drawn for the crown and invert using as ordinates $\frac{H - P_x}{H - P_{11}}$, and as abscissas $\frac{d_x}{d_{11}}$, where H is the head above elevation 1036.67; P_x is the pressure at any piezometer in the bellmouth referred to elevation 1036.67 as a datum; P_{11} is the pressure at the reference piezometer, number 11, referred to the same datum; d_x is the distance from the face of the bellmouth to any piezometer; and d_{11} is the distance from the face of the bellmouth to piezometer number 11, as shown at the bottom of figure 6. Values of $\frac{H - P_x}{H - P_{11}}$ express the pressure drop at any piezometer as a percent of the total, while the distance ratio $\frac{d_x}{d_{11}}$ eliminates any small differences between the location of piezometers in the model or prototype. Two of the prototype tests were omitted from the diagrams so as not to confuse them with more superimposition of the points, but all of the model values are included. The actual pressures at the piezometers in the bellmouth were all positive (above atmospheric) except for low heads, and these did not exceed a vacuum of 10 feet of water.

The prototype curves illustrate that the pressure drop, in percent, is practically constant at nearly all the piezometers regardless of the head. This is not true for the model tests, even though the average values of the model are within approximately 5 percent of the prototype. Reasons for the dispersion of the model data may be explained by experimental error, different approach conditions, and unstable flow in the model at lower Reynolds numbers, although above critical Reynolds number.

Experimental errors are probably due to the installation of the model piezometers, even though great care was taken to install them properly. Nevertheless, any minute obstructions on the boundary surfaces and in the piezometer openings have serious effects. The laboratory experiments, moreover, were made under considerable duress to enable the design to keep ahead of construction in the field. Consequently, the customary close checks on the data could not be made as complete as they would have been under a more normal procedure. The different approach conditions refer to the absence in the laboratory of a trashrack upstream from the bellmouth; however, it was observed from separate tests on the bellmouth that the trashrack effects were negligible.

Evidently the differences in friction play a small part even though a comparison is confined to the bellmouth alone. No estimate can readily be made of the relative roughness of either the model or prototype surfaces, but since the type of surface in each is nearly the same, the relative roughness of the model would be greater. Consequently at lower Reynolds numbers, the resistance factor for the model will vary considerably more within its range of tests. It can be further assumed that the model tests were for rough flow, whereas the prototype tests were more nearly smooth flow. Finally, it might be added that a transition from a laminar sublayer to a turbulent sublayer will develop less rapidly in the model, thus influencing the velocity distribution and pressures. It is reasonable to assume, therefore, that the model flow is not sufficiently stable to give the consistent data obtained for the prototype. Accordingly, it is believed that the laboratory tests should have been accomplished at greater velocities (higher Reynolds numbers), disregarding heads based on the scale ratio of the model. Had this been done, the model data as analyzed on figure 6 would have been more consistent and would have approached the prototype values more closely, although 5 percent is certainly good agreement for the average curves.

Pressure drop in elbow and cone. Dimensionless curves have been drawn in figure 7 to show the pressure drop in the elbow and cone in a manner similar to that described for the bellmouth comparison. To eliminate the differences in friction up to the elbow, the pressure drop has been expressed as $\frac{P_{23} - P_x}{P_{23} - P_{33}}$ where P_{23} and P_{33} are the pressures referred to elevation 1021.67 as a datum, at the start of the elbow and at the end of the cone, respectively; P_x is the pressure at any piezometer referred to the same datum; d_x is the distance along the center-line from piezometer 23 to any piezometer; and d is the center-line distance from piezometer 23 to piezometer 33, as shown at the bottom of figure 7. As in figure 6, two prototype tests have been omitted for clarity, but all model tests are shown. The heads, however, are referred to elevation 1021.67 instead of elevation 1036.67, and average center-line pressure drops have been added. The reason for negative ordinate values for the pressure drop on the crown is due to an increase in pressure from piezometer 23, thus making the numerator negative in the ratio $\frac{P_{23} - P_x}{P_{23} - P_{33}}$.

Although not shown on the figure, the center-line pressure drop falls as a good average between the curves for the crown and invert for both model and prototype. This may be seen from the curves given on figure 5.

Figure 7 shows the data for both the model and prototype as being almost equally consistent, the model values still being slightly erratic. Reasonable agreement, however, is lacking. In general, the pressure drop indicated from the model tests is about 30 percent greater than is shown from the field measurements. This unsatisfactory agreement may be explained by the principal reasons applied to the bellmouth comparison; namely, experimental errors and unstable flow for the range of Reynolds numbers in the model.

In computing the model data for the elbow and cone, it was evident from the excessive dispersion of the points that some of the pressure readings were in error. To adjust these, a curve of head versus pressure was plotted for each piezometer. Such curves should be a straight line, so it was possible to detect which readings were erroneous and to adjust them to the line. Unfortunately, if the pressures at the start of the elbow, P_{23}, and at the end of the cone, P_{33}, are not initially correct or are not properly adjusted, then the total pressure drop $P_{23} - P_{33}$ causes an error in the ratio $\frac{P_{23} - P_x}{P_{23} - P_{33}}$ plotted in figure 7.

One of the field tests was similarly adjusted where obvious errors were noted. By checking the piezometer numbers attached to each header, it was found that the erroneous readings were all in a group located in one piezometer header. It was concluded that the pressure from another piezometer in the same header was being registered at the time this group was being recorded. An error of this nature is easily made when so many piezometers are required, unless particular care is taken to be certain that only one piezometer is being read at a time.

Since the length of the elbow and cone is nearly six times that of the bellmouth, it is evident that the differences between the model and prototype friction will have still more effect on the comparison, even though a study of the losses shows the bend loss or centrifugal effect to be considerably greater. The latter seems to be noticeable in the prototype curves on figure 7, where the points are more scattered than for the bellmouth measurements shown on figure 6. Time did not permit a more complete examination of the data for the elbow and cone which might disclose a more valid method for comparing the pressure drop.

Discharge measurements. The model and prototype discharge comparison is indicated on figure 9. Curve A is based on the 1 to 17 scale model tests for which $Q_p = Q_m \cdot L_r^{5/2}$, while curve B is the model rating curve corrected to take into account the difference between model and prototype friction. To make this correction, it was first necessary to estimate an average friction coefficient for the prototype outlet. This was not readily accomplished because of a lack of reliable friction coefficient data at Reynolds numbers as high as 5×10^7; nevertheless, it was assumed that a value of 0.0085 for the friction factor would be reasonably accurate. The model friction coefficients and Reynolds numbers were then computed from the laboratory pressure tests and plotted on figure 8. Since the model friction is too large, the model heads for given discharges would be too high. Accordingly, assuming all losses may be scaled to the prototype except friction, the model heads were reduced by the value $(f_m - f_p) \frac{l}{d} \cdot \frac{V^2}{2g}$, in which f_m was obtained from figure 8 and $f_p = 0.0085$ as assumed. After the pressure data for the prototype became available, the friction coefficients and Reynolds numbers were computed and plotted on figure 8. The average value for f_p was 0.0095 as compared to an estimate of 0.0085. Two 40-foot reaches were used for the prototype computations, starting at a distance of approximately eight diameters downstream from the entrance of the outlet, corresponding reaches being used for the model computations. That this is probably not a sufficient length to develop a normal velocity distribution is seen on figure 8, where the upper points were obtained using the first 40-foot reach, while the lower points were obtained from the following 40-foot reach.

The discharges given by curve C, which were used for computing the prototype friction factors and Reynolds numbers, was developed from prototype pressure measurements, treating the bellmouth as a submerged orifice. The average pressure heads measured at piezometer ring 11 enabled the values of h_e to be computed (see diagram on figure 9). The four piezometers at this point are located 4 feet 5-3/16 inches from the entrance and at the tangent between the bellmouth and transition immediately downstream, as shown in section on figure 5. Using the area of the bellmouth at this point, and assuming a coefficient of discharge $C_q = 0.98$, the orifice formula $Q = 0.98 A_{11} \sqrt{2g\,h_e}$ was applied. Similar results would be obtained using h_e and the area at the start of the bellmouth, but with $C_q = 0.60$, or applying the same area and coefficient with the head on the bellmouth reduced by the losses in the outlet instead of h_e.

Curve D has been obtained from discharge measurements at a river gaging station one-half mile downstream from the dam. Since it is impossible to pass the flow of the Columbia River through only one outlet, the discharge applicable to a single outlet had to be computed by an indirect method. First, a rating curve was obtained for one lower outlet before the water surface in the reservoir reached the intermediate outlets. This was accomplished by plotting effective head (reservoir elevation minus tailwater elevation at the gaging station), against the discharge for one lower outlet (total measured discharge divided by the number of lower outlets operating). As soon as the intermediate outlets began to operate, the total river discharge increased accordingly, but knowing the effective head on the lower outlets and the number operating, it was possible to obtain the discharge per outlet from their rating curve and, thus, the total discharge contributed by the lower outlets. Subtracting this value from the measured flow in the river and dividing by the number of intermediate outlets operating, gave the discharge for one outlet as shown by curve D. This method assumes that each outlet will discharge the same amount, and that the jets from the intermediate outlets will not affect the effective head on the lower outlets. Although these assumptions are not strictly correct, it is believed that curve D, and certainly curve C, would agree within 5 percent of any rating curve obtained by measuring the discharge of only one outlet.

In comparing the four rating curves over a range of head of from 230 feet to 50 feet, the uncorrected model discharge (A) is from 10 percent to 21 percent less than the prototype discharge computed at the bellmouth (C), while the corrected model discharge (B) is only 3 to 8 percent less. Similarly, the uncorrected model discharge (A) is from 11 to 27 percent less than the measured prototype discharge (D), while the corrected model discharge (B) is only 5 to 15 percent less.

SUMMARY

The agreement between the model and prototype in this quantitative comparison is reasonably good except in one instance. Considering experimental errors and lack of similitude relative to Reynolds number, the pressure drop in the model bellmouth agrees with the prototype within 5 percent; the pressure drop in the elbow and cone of the model, on the other hand, is 30 percent greater than in the prototype; and, finally, the model estimate of the prototype discharge is, on an average, 8 percent low. Even though complete accord could not be obtained, the 1 to 17 scale model estimates are on the safe side and the field measurements show that an excellent prototype design has been evolved. It follows, therefore, although complete similitude was lacking, that the model could be relied upon to prove not only the inadequacy of several proposed designs, but also to develop the satisfactory design finally adopted.

CONCLUSIONS

To better accomplish model-prototype comparisons, it is evident that plans should be made for procuring field measurements during the model tests and while the prototype is being designed. This not only enables a sufficient range of model data to be obtained, but it also makes certain that the instruments required at the prototype will be properly located and incorporated as a part of the design of the structure. Even these precautions may not yield the desired results because of the difficulties in actually obtaining prototype flow measurements. Thus, the reasons for so few quantitative comparisons is traced to the necessity of waiting years before sufficient water is available, or to the difficulties in measuring large discharges and high velocities, or to the fact that the measured discharge is derived from several sources instead of the one to be used in a comparison. Finally, the model experiments themselves introduce objectionable factors influencing comparisons, the chief one being the inability to test the model of conduits at the correct range of Reynolds numbers. In any event it is important to build hydraulic models to as large a scale ratio as possible.

The qualitative and quantitative examples of model-prototype comparisons given in this paper reaffirm the high degree to which the model can be relied upon. Although the avowed purpose of any comparison, particularly the quantitative type, should be to affirm the theory of hydraulic models or to modify the transference relations if necessary, it cannot be accomplished from only a few comparisons. At present, the stress should be laid on the limitations and accuracy of quantitative estimates based on model tests. In this regard, it is equally important to show both inferior and good model-prototype comparisons. The lessons learned therefrom will certainly demonstrate the adequacy of present-day model research.

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Those who are closely associated with model-prototype work realize the few references that are available on the subject. Accordingly, a bibliography of this work is added to include all available information in this symposium. The list, including references in the text, is probably far from complete, but it represents all that could be found by the junior author during his travels as the Society's Freeman Scholar in 1940-41. Acknowledgment is made to Professor E. W. Lane, University of Iowa, and to the U. S. Waterways Experiment Station, Vicksburg, Mississippi, for many of the references.

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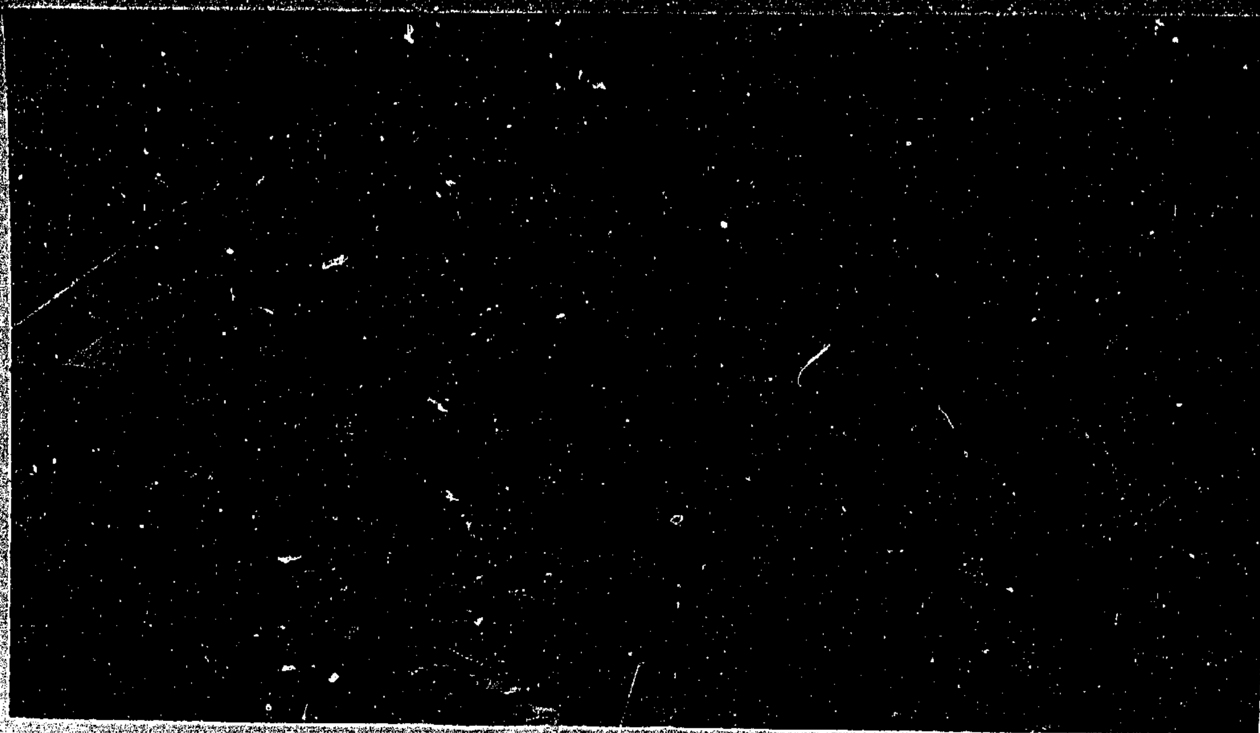
J. E. Warnock
H. G. Dewey, Jr.
January 12, 1942



1 to 15 Scale Model

Prototype (Sunnyside Main Canal)
Fig. 1. - Check Drop Before Revision - Discharge 1,300 Second-Feet

1 to 15 Scale Model

Prototype (Sunnyside Main Canal)
Fig. 2. - Check Drop After Revision - Discharge 1,300 Second-Feet.

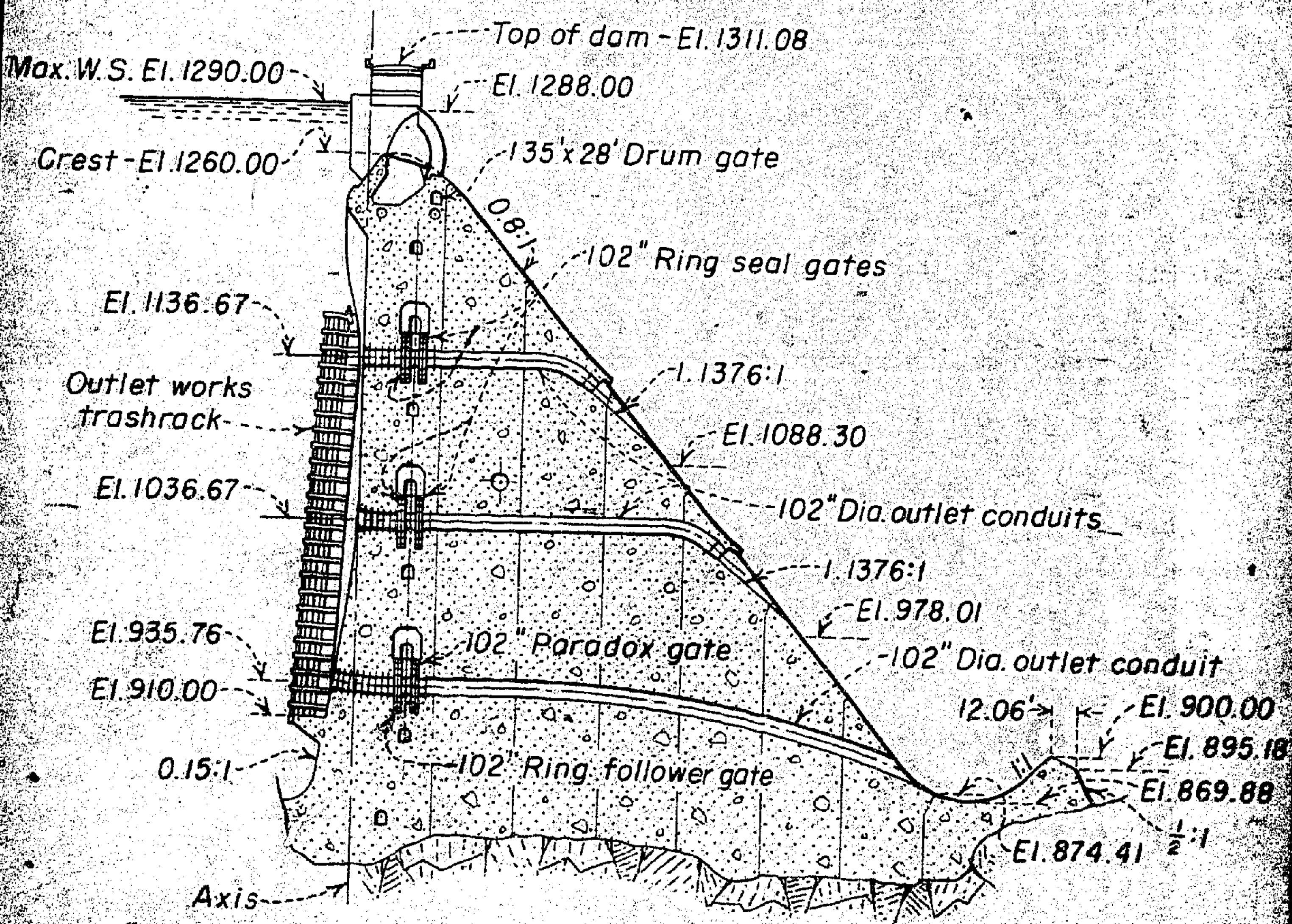


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FIG. 3 SPILLWAY SECTION - GRAND COULEE DAM

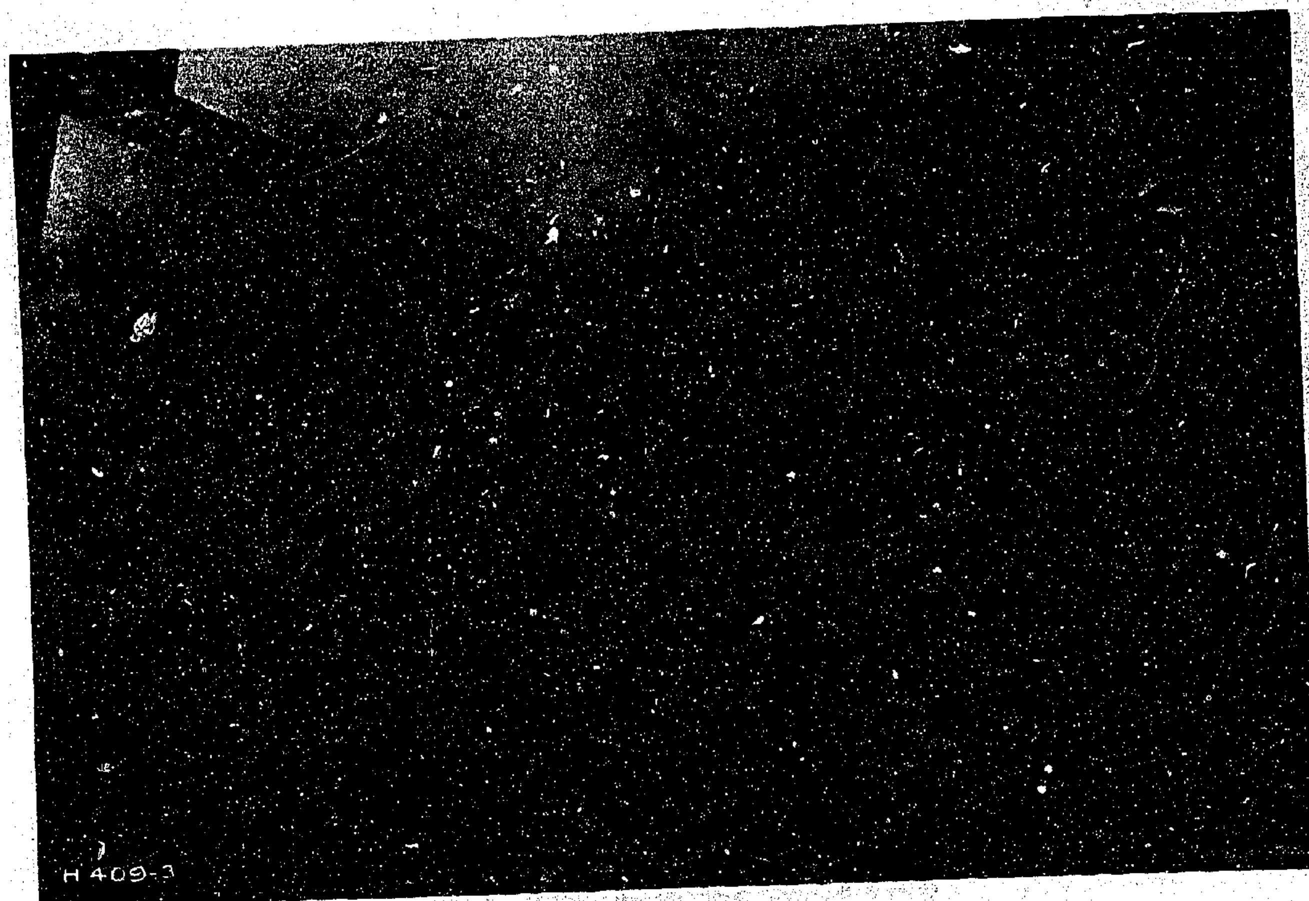

Figure 4 - Upper Outlet - Discharge 2,400 Second-Feet.

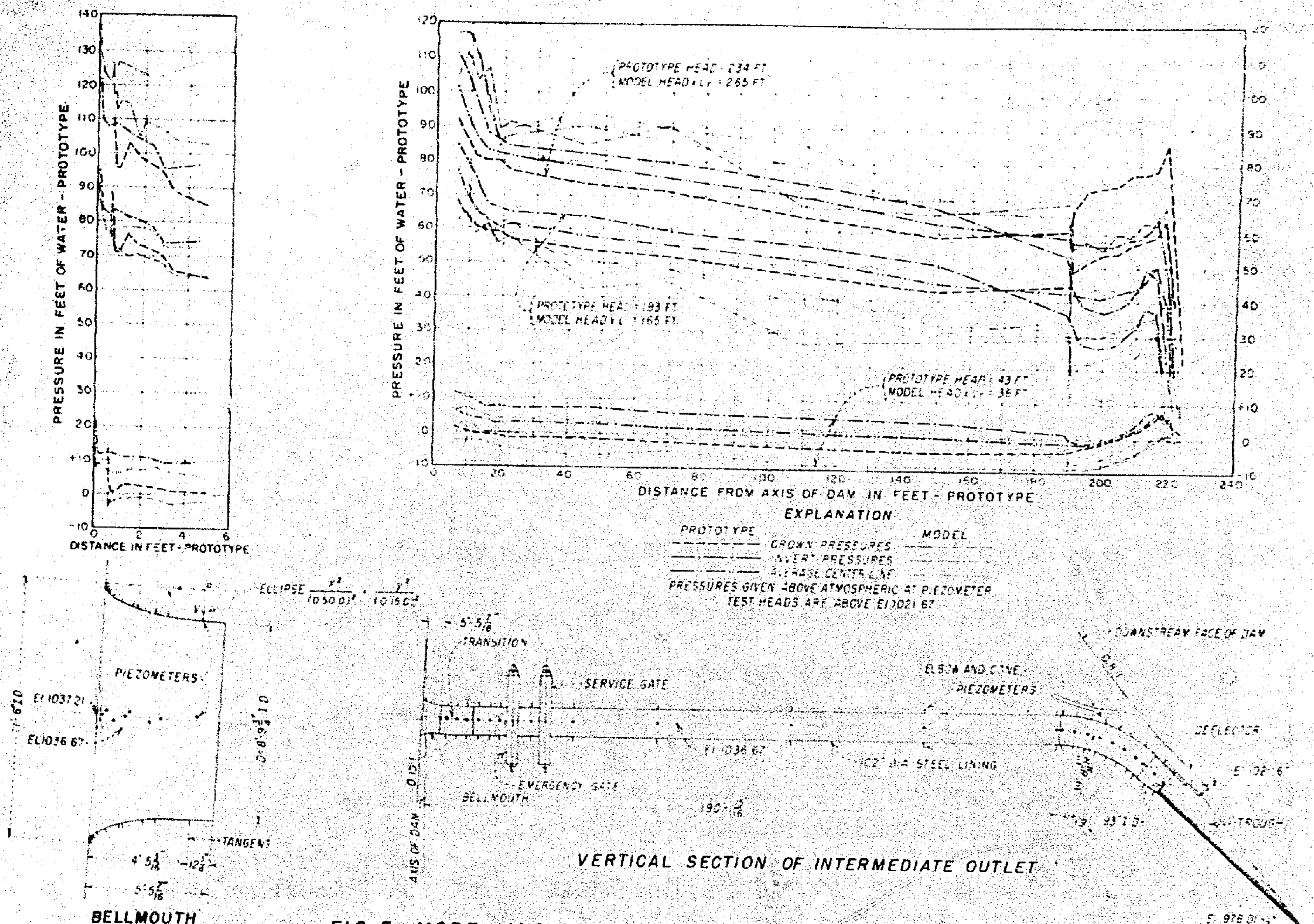

FIG. 5 - MODEL AND PROTOTYPE PRESSURE CHARACTERISTICS

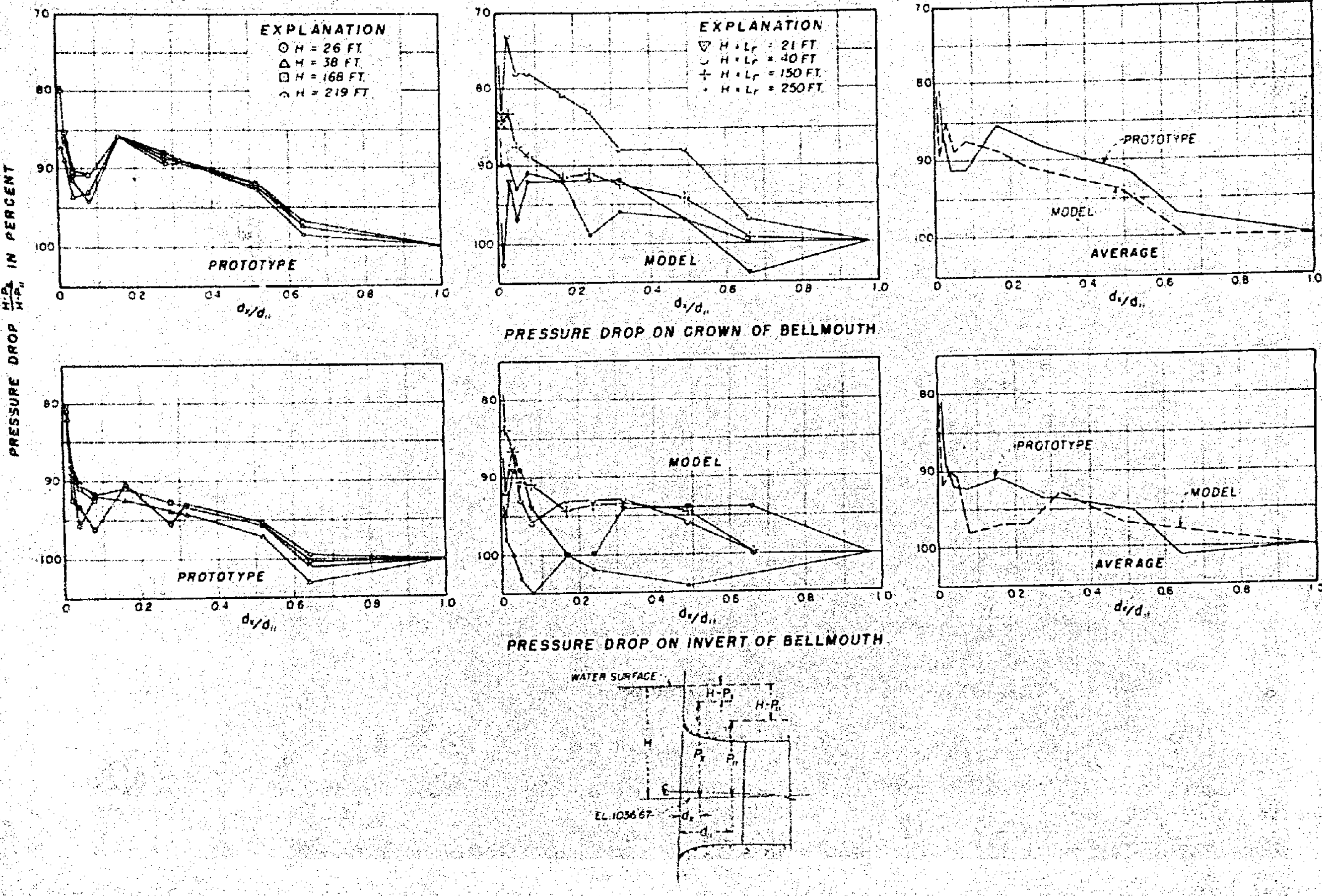

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FIG. 6 - COMPARISON OF MODEL AND PROTOTYPE PRESSURE DROP IN BELLMOUTH

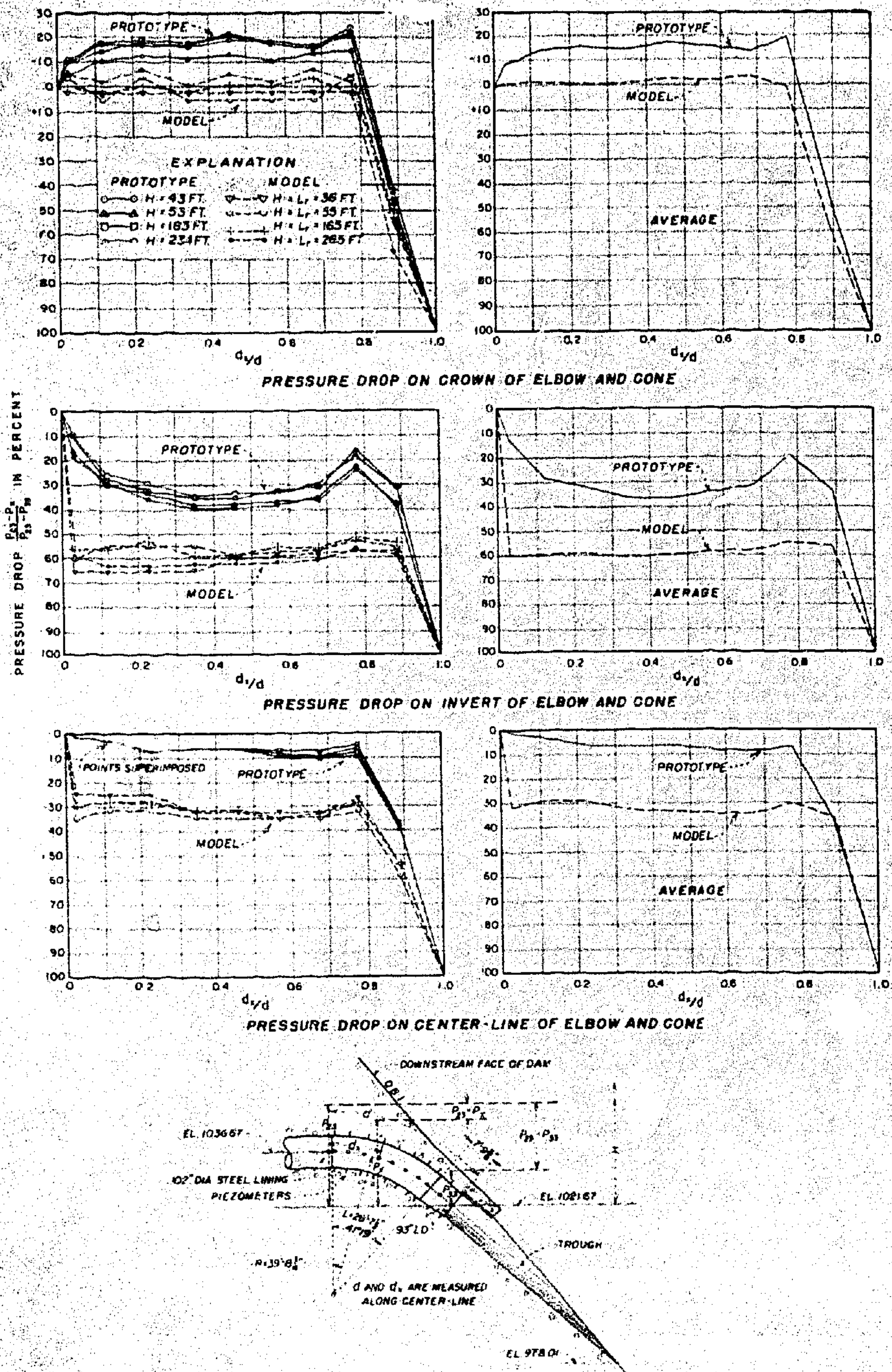

FIG. 7 - COMPARISON OF MODEL AND PROTOTYPE PRESSURE DROP IN ELBOW AND CONE

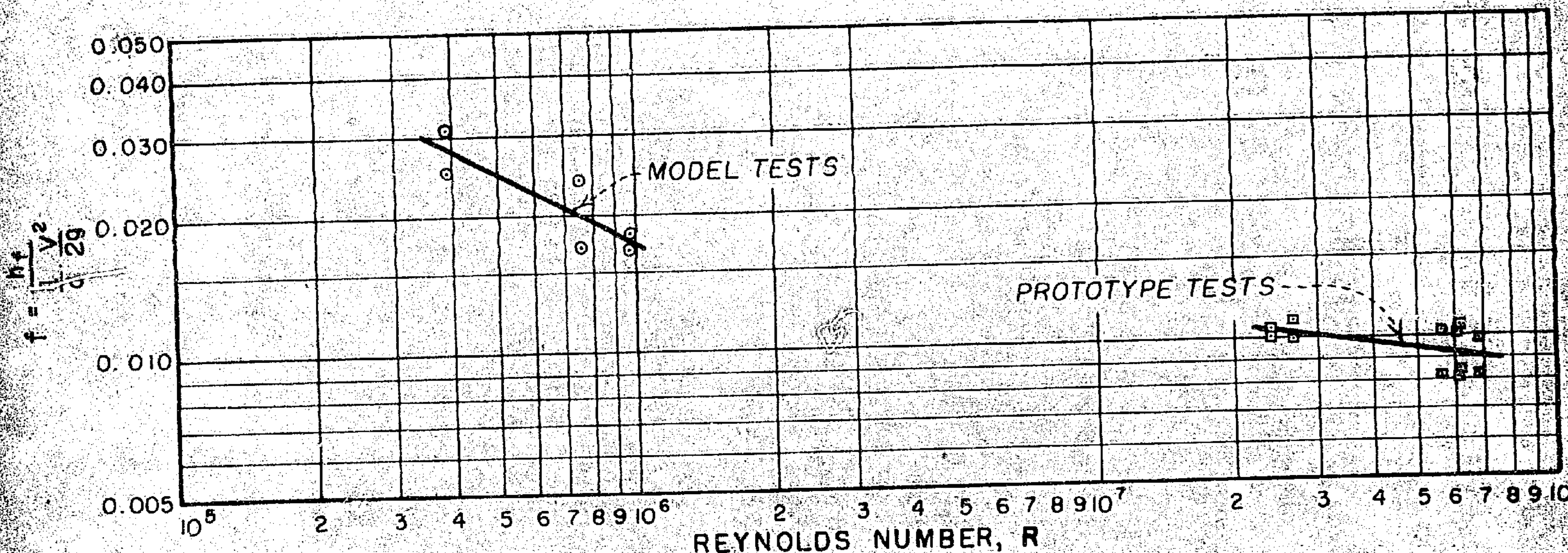

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FIG. 8–MODEL AND PROTOTYPE COEFFICIENTS OF FRICTION

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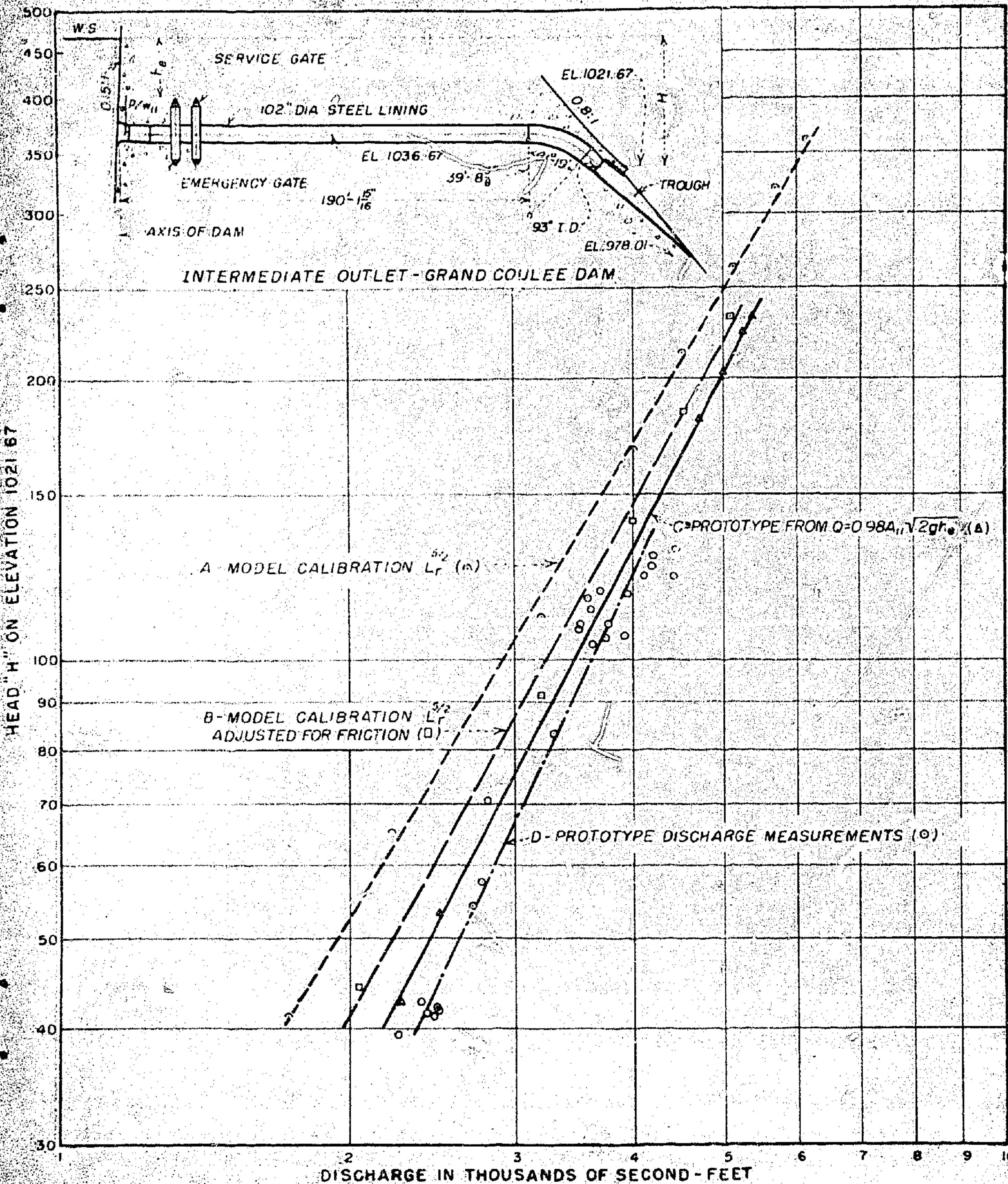

FIG. 9. - COMPARISON OF MODEL AND PROTOTYPE DISCHARGE

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