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HYDRAULIC LABORATORY STUDIES OF A 4-FOOT-WIDE WEIR BOX TURNOUT STRUCTURE FOR IRRIGATION USE

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Engineering and Research Center
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16. ABSTRACT

A hydraulics laboratory study helped develop the design of a 10-cfs-capacity irrigation weir box structure and determined the head-discharge relationship. A baffle arrangement was developed to adequately distribute the inflow from a circular pipe and provide satisfactory approach flow conditions to the 4-foot-wide measuring weir. Discharge rating calibration was obtained for flows between approximately 2 and 12.5 cfs. The calibration did not agree with accepted equations for suppressed rectangular weirs because of irregular approach flow conditions and the method of measuring head. A discharge equation was derived by combining the Kindsvater-Carter method with a variable effective head concept which considers the disturbances in the approach flow.

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September 1972

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Division of General Research
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Denver, Colorado

UNITED STATES DEPARTMENT OF THE INTERIOR * BUREAU OF RECLAMATION
Rogers C. B. Morton, Secretary
Ellis L. Armstrong, Commissioner

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INTRODUCTION

Irrigation turnout structures with a 3-foot (0.914 meter) wide suppressed rectangular weir for measuring discharges up to about 5 cubic feet per second (cfs) (about 140 liters per second (l/sec)) have been in use for a number of years on the Columbia Basin Project in Washington. These turnouts have proven to be economical to build, easy to operate, accurate and reliable as flow measuring devices, and generally less troublesome than some other measuring structures.

The geometry of these weir boxes (and accessories) was developed during full scale hydraulic model testing in the Bureau's Engineering Laboratories in Denver.[1] The general weir box concept, however, had evolved from older structures in use on the Yakima Project, Washington, which utilized Cippoletti Weirs for flow measurement. Although the geometry and flow-measuring characteristics of the two types bear little resemblance, the newer structures on the Columbia Basin are still commonly (if inappropriately) referred to by operators as "Yakima weir boxes."

Weir box turnout structures having a greater capacity than those in use on the Columbia Basin were required for turnouts from new laterals of the Wahluke Branch Canal on Block 25, Columbia Basin Project, near Mattawa, Washington. A preliminary design was prepared for a 10-cfs (about 280-l/sec) capacity weir box. The dimensions were scaled up from the existing 5-cfs weir box so that the velocities in the approach flow to the measuring weir would be approximately the same. The width of the weir and box was 4 feet and the geometries of the baffles, weir gage stilling box, and weir were similar to those of the 5-cfs turnout structure. A 24-inch-diameter inlet pipe was proposed.

The Bureau design employs vertical baffles close to the inlet pipe and a stilling box across the total width of the structure. The stilling box is immersed in the flow slightly below the minimum water surface (the weir crest elevation). The upstream face of the stilling box acts as a support for the upper end of the baffles. These appurtenances permit a considerable shortening of the weir box by helping to distribute the inflow from the pipe and smooth the water surface. The desired design employing the above features would be the shortest box that would provide stable flow over the weir and yield a unique head-versus-discharge relationship. However, since the approach flow to the weir would still be rather rough, the head/discharge relationship would likely be different from the standard for suppressed rectangular weirs, and therefore it cannot be predicted accurately by analytical means.

Considerable information is available in engineering literature on the performance and discharge rating of suppressed rectangular weirs.[2,3] The discharge rating equations can be applied to a large range of weir sizes and operating heads if the weir is installed according to standard conditions. Two important requirements must be satisfied. First, the approach channel to the weir must be long enough to insure normal velocity distribution for all discharges; second, the head must be measured at a point where the water surface has not been influenced by drawdown or obstructions in the approach channel. Since these conditions are not met in the Bureau-designed weir box turnouts, the standard discharge rating equations do not apply.

Laboratory studies of a full scale turnout structure were undertaken to improve the hydraulic design and calibrate the weir box for a capacity of 10 cfs.

LABORATORY TEST SUMMARY

A satisfactory baffle arrangement to distribute and smooth the flow approaching the measuring weir was developed through trial-and-error testing. The baffle opening area was 1.40 times the inlet pipe area. A stilling box across the entire width of the weir box proved to be a satisfactory means of measuring a characteristic head for discharge rating purposes.

A discharge rating calibration for the final recommended geometry was obtained for flows between about 2 and 12-1/2 cfs. The measured discharge was always higher than that computed by two well-known equations for standard suppressed rectangular weirs, the Francis and Kindsvater-Carter formulas. The difference may be attributed largely to disturbances created in the approach flow by the baffles and weir gage stilling box, and the manner of measuring the head.

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[1] W. P. Simmons, "Hydraulic Model Studies of Small Weir Box Turnout Structures for General Irrigation Use," Report No. HYD-396, USBR, Denver, Colorado, 1954.

[2] "Water Measurement Manual," USBR, Second Edition, Chapter II, Denver, Colorado, 1967.

[3] H. W. King, and E. F. Brater, "Handbook of Hydraulics," Fifth Edition, Section 5, McGraw-Hill, New York, 1963.

A discharge equation was derived by combining the Kindsvater-Carter method with a "variable effective head" concept which takes into account the disturbances in the approach flow.

APPLICATIONS

The weir box was specifically developed for use at turnouts from laterals of the Wahluke Branch Canal on Block 25, Columbia Basin Project. The same geometry used in the laboratory has been included in Specifications No. DC-6624, Supplemental Notice No. 2, Drawing No. 222-D-22481 (see Figure 6). The discharge calibration obtained in the laboratory has been used in preparing the "Discharge Table" included in the above specifications drawing.

The weir box can be used where flows up to about 12-1/2 cfs from a pipe outlet are to be measured. Either the discharge table in Figure 6 or Equations 11 and 12 can be used for prediction of flow quantities. If the baffle arrangement is altered, however, the calibration is also likely to change.

LABORATORY INSTALLATION

Description of the Model

The full scale model of the preliminary design was constructed using surface-treated plywood with appropriate metal appurtenances where required (Figures 1 and 2). Important dimensions are shown on the drawing in Figure 3.

The stilling box had two triangular holes at the bottom of the downstream face to allow passage of water into the well. Inside the well, a weir gage was set with "zero" at the same elevation as the top of the weir blade. The head indicated by this gage was used for the weir discharge rating. The water surface was also measured by means of a piezometer tap located in the floor of the weir box, 3.5 feet downstream from the entrance, and about 3 inches from the right wall. The piezometer was connected by flexible tubing to a 4-inch (10.16-centimeter (cm)) diameter stilling well in which the water surface elevation was measured with a graduated hook gage.

The inlet pipe diameter of 24 inches (0.610 meter) was attained by two step increases from 12-inch-diameter pipe. A 12-inch gate valve, located about 28 feet upstream from the box inlet was used to control water flow into the model. Three right-angle bends and the gate (normally throttled) through which the flow had to pass before it entered the inlet pipe could have caused unsymmetrical and spiraling flow at the entrance to the weir box over at least a portion of the total flow range. Entrance conditions in a typical field installation could also be far from ideal, and could produce extremely irregular, if not similar, velocity distribution in the flow as it enters the weir box. The model inlet flow conditions, while not modeled after a specific structure, could represent a typical field installation.

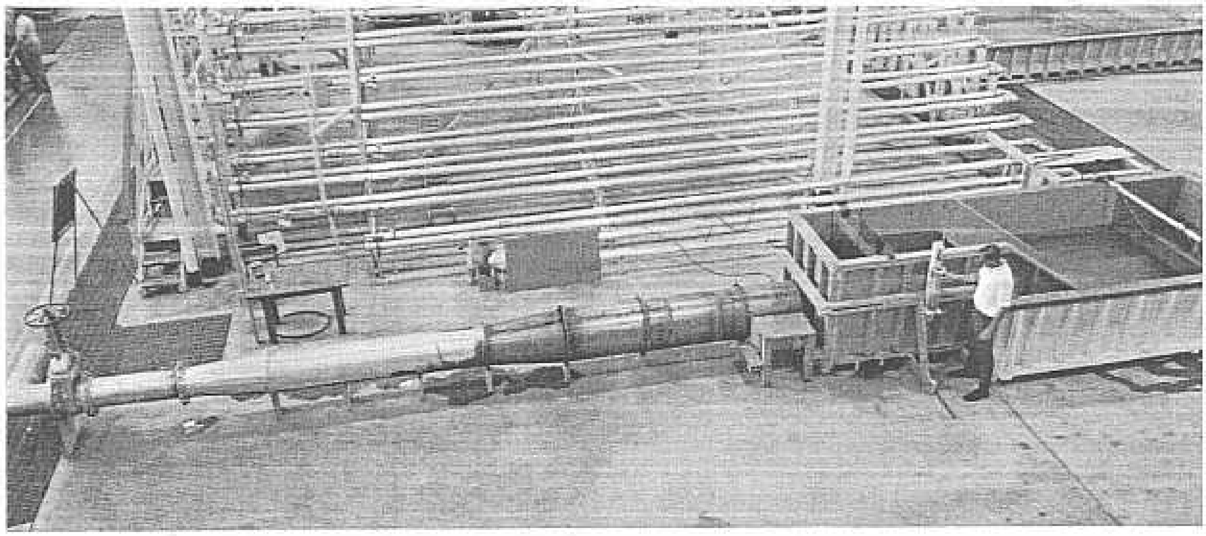


Figure 1. Laboratory installation. Photo PX-D-71971

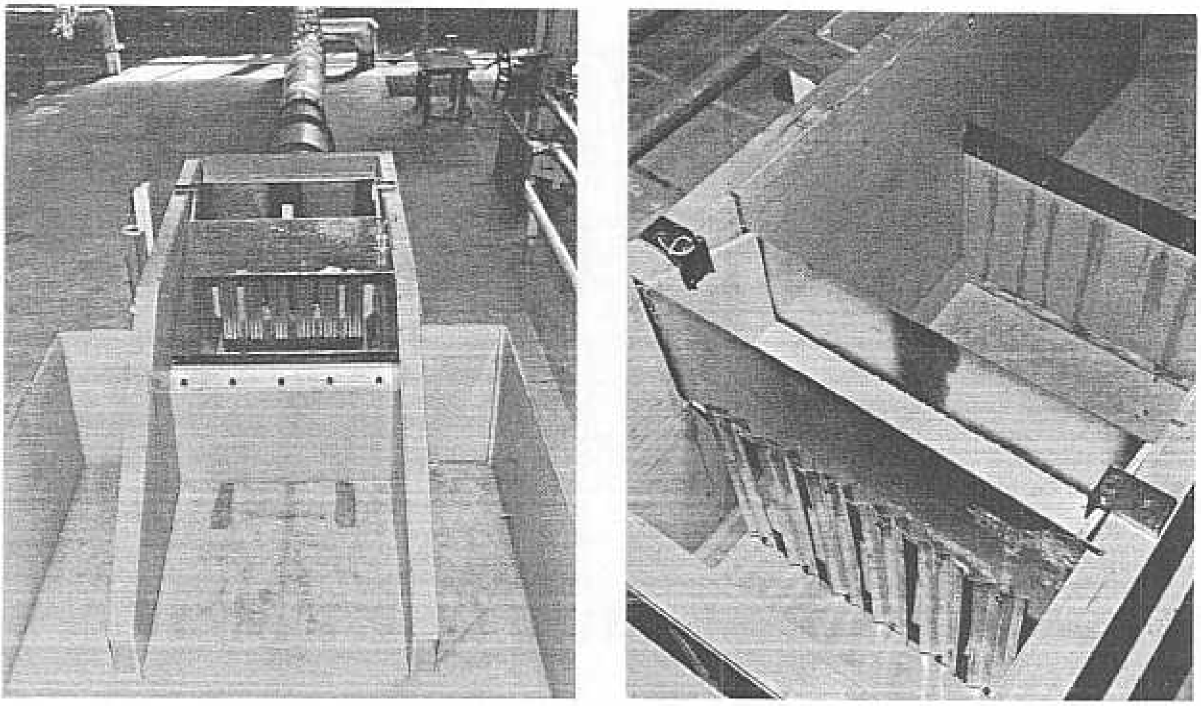


Figure 2. Weir box details with preliminary baffles. Photos PX-D-71972 and PX-D-71973

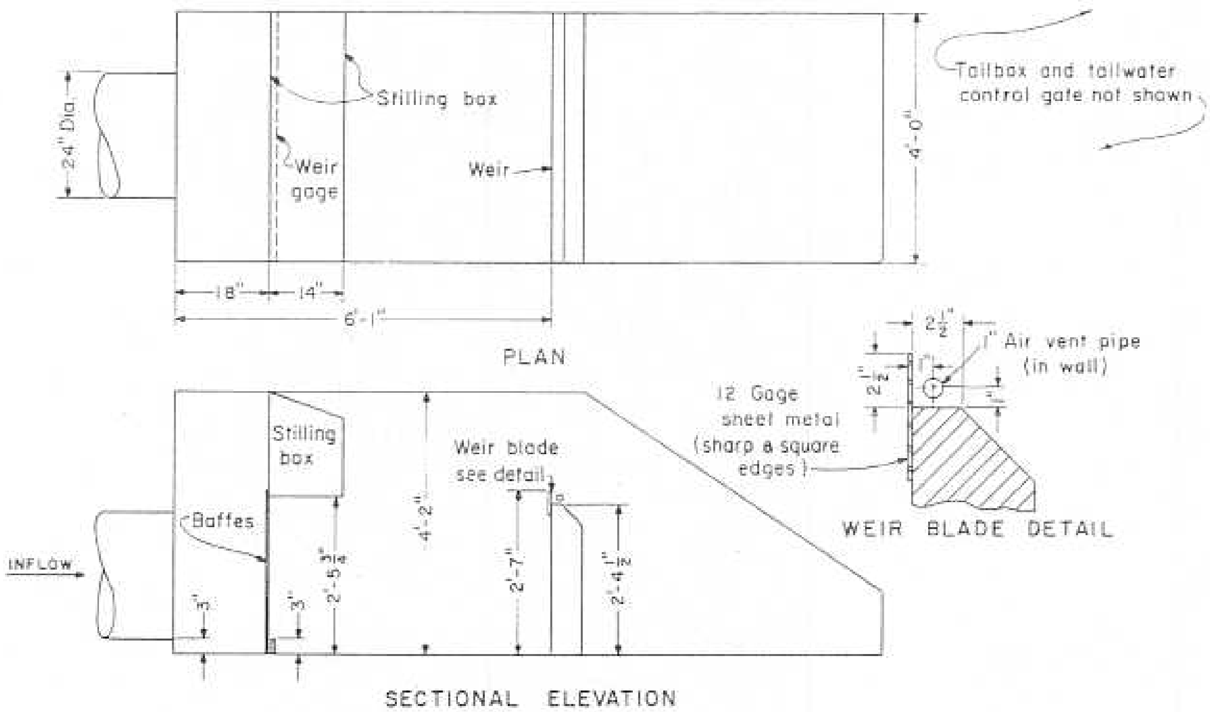


Figure 3. Laboratory installation.

A wide tailbox was connected around the downstream end of the weir box. The tailbox had a movable tailgate to allow for adjustment and control of the tailwater elevation below the weir.

Water Supply and Measurement

Water was supplied to the model through the laboratory's permanent recirculating system. The flow was measured with permanently installed volumetrically calibrated Venturi meters. Three different meters were used to calibrate the weir. Differential head across the Venturi meters was measured with pot-type mercury manometers. This system is generally considered to be capable of measuring the discharge with an error of less than 1 percent.

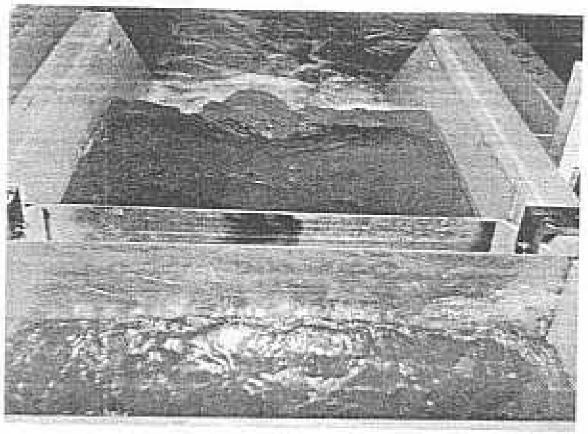
THE INVESTIGATION

Preliminary Design

The flow through the weir box, with the preliminary baffle arrangement shown in Figure 2, was rough and turbulent for discharges greater than 5 cfs, and extremely rough and unsteady at a 10-cfs discharge (Figure 4). One to two feet upstream of the weir the axial velocity of the water was much less in the center of the box than near the sides. At times the center velocity had a component upstream. A large boil resulted in the water surface near the weir, slightly to the left of the centerline (looking downstream). The location of the boil shifted, causing fluctuations of the water surface in the weir gage stilling well.

A water surface drop of about 0.6 foot (18 cm) resulted in the flow through the baffles. The baffle openings apparently acted as orifices. Their area was 2.83 square feet (sq ft) (0.263 m^2) compared to an area of 3.14 sq ft (0.292 m^2) for the inlet pipe. The flow contracted as it passed through the baffle slots, producing jets with velocities higher than the inlet pipe velocity. These jets interacted and dissipated in a manner which caused extreme eddying action, producing a rough and irregular water surface.

The flow through the weir box was judged to be unsatisfactory. Modification of the baffles was considered as the first step in improving flow conditions. Other possible changes, such as lengthening and/or widening of the box, were to be made only if baffle modification did not produce satisfactory flow conditions.



Q = 7 cfs

Q = 10 cfs

Figure 4. Flow through weir box with preliminary baffles. Photos PX-D-71975 and PX-D-71974

Criteria for evaluation of modifications were established. For any modification, the flow conditions at 7-cfs discharge were to be evaluated first. If the flow appeared satisfactory, 10-cfs discharge would then be checked. Rough (but steady) flow conditions would be acceptable at the latter discharge. The satisfactory arrangement would then be tested at higher flows to find the upper limit of steady flow conditions.

For the preliminary design and all subsequent modifications, the flow conditions were evaluated qualitatively. Relative water surface roughness was judged by observation and from photographs. Velocity distribution and direction was sensed at various points in the approach flow. For the final design, velocity traverses were performed at several cross sections using a pygmy current meter.

Modifications with Preliminary Baffles

A different arrangement of the same six corrugated metal baffles was tried. The side slots were reduced to 1 inch in width, and more of the open area was concentrated toward the quarter points of the box width, while the center slot remained about 1.6 inches wide. The velocity distribution did not change appreciably from the preliminary design. The velocity was high along the sides, low (and sometimes in the upstream direction) in the center, and a boil was produced.

Another baffle arrangement was tried using only five of the 5-1/3-inch-wide baffles. The side slots were 1 inch wide, while the other four openings were about 4-3/4 inches wide. The ratio of baffle opening area to pipe area was 1.24. The flow conditions were improved. Velocities were much more evenly distributed downstream of the stilling box. A center boil started forming intermittently at about 7-cfs discharge, and at 10 cfs the boil was smaller than in previous arrangements. An intermittent vortex formed near the center of the weir.

The above arrangement did not produce completely satisfactory flow conditions. The testing up to this point suggested that narrower baffles, with an opening area greater than the inlet pipe area, would distribute the flow more uniformly.

No Baffles

Before installing any new baffle arrangement, flow through the weir box without baffles was observed. The highest velocity was in the center through the total length of the box. At higher flows, boils formed on the water surface at both walls. At the weir, the water surface was much higher at the walls than in the center. At 10-cfs discharge the flow was extremely rough.

Three-inch-wide Baffles

Six different arrangements using 3- by 3/8-inch (7.62- by 0.95-cm) plywood baffles were tried. The number of baffles used in different schemes were 9, 8, and 7, with corresponding ratios of baffle opening area to inlet pipe area of 1.22, 1.40, and 1.58. With all arrangements the flow distribution downstream of the baffles was more uniform than had been with the preliminary design baffles.

The scheme which produced the most satisfactory flow had 8 baffles. There was no space between the walls and adjacent baffles. The center opening was 4 inches wide, while the remaining openings were about 3-1/3 inches wide. At 7-cfs discharge, small boils formed intermittently near the center at the weir. With 10 cfs discharging through the box, the flow was acceptable. The velocity distribution was relatively uniform and symmetrical, producing only small random boils which occasionally combined to form one larger boil. The larger boil eventually broke up, and at times the entire water surface was quite smooth.

As a result of the reduction in width of each baffle the flow distribution characteristics of the total baffle screen were improved. This suggested that a further reduction in baffle size might produce even smoother flow conditions.

Two-inch-wide Baffles

Six arrangements with 2- by 3/8-inch (5.08- by 0.95-cm) plywood baffles were evaluated. In all cases, 12 baffles were used, with a baffle opening area to inlet pipe area of 1.40. In all schemes, the outermost baffles were adjacent to the wall, while the middle baffles were shifted to evaluate small variations in their spacing on the velocity distribution at the weir. All arrangements produced good velocity distributions and acceptable water surface characteristics with only irregular and intermittent boil formation at the weir.

Final Baffle Arrangement

The most practical 12-baffle scheme with good flow conditions had a center opening 3-1/2 inches (8.89-cm) wide, with remaining openings 2 inches (5.08-cm) wide (Figure 5). The side baffles were placed 1/4 inch from the wall to facilitate installation. This scheme was recommended for the final weir box design.

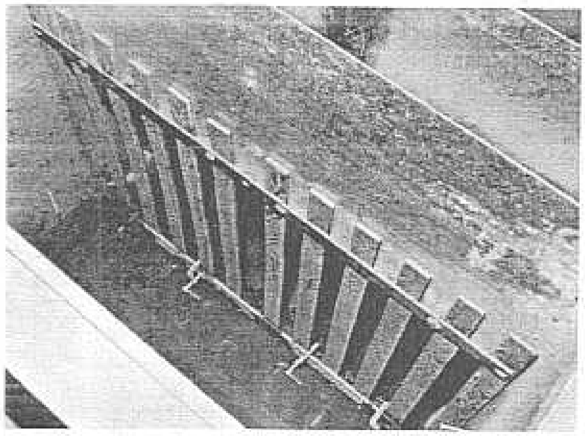


Figure 5. Recommended baffle arrangement. Photo PX-D-71976

After the recommended baffle scheme was developed, no other changes in the length or width of the weir box were deemed necessary. The dimensions of the recommended weir box can be seen in Figure 6, which is a specifications drawing of a typical installation of the weir box on Block 25 of the Columbia Basin Project.

The flow through the weir box appeared satisfactory up to about 13 cfs (370 l/sec). Flow conditions at several discharges can be seen in Figures 7, 8, 9, and 10. At 13-cfs discharge the water surface became rather rough and could be seen to surge, although no persistent boils were present. The surging was also reflected in the weir gage stilling box by fluctuation of the water surface of about 0.02 foot (0.6 cm) at a constant discharge. When calibration of the weir was performed, irregularities in the head-versus-discharge relationship could be noted slightly above 12-cfs discharge.

Velocity measurements were performed using a pygmy current meter at several cross sections for 7-cfs discharge, and near the weir crest for about 13-cfs discharge. The measurements indicated that the flow was fairly well distributed as it flowed over the weir.

Velocities were not measured in the inlet pipe, but the distribution probably was not symmetrical due to the approach pipe and control valve geometry. It is believed that the velocity distribution through the box was usually influenced by some inlet flow assymmetry and irregularity. The velocity distribution at the weir in a typical field installation could be different, but probably no more severe than existed in the model.

The water surface drop between the inlet and the stilling well was measured at the center, walls, and quarter points, Figure 11. The total energy head loss across the pipe inlet, baffles, and weir box stilling well was computed as the water surface drop plus the difference of the velocity heads of average inlet pipe velocity and the average velocity downstream of the baffles. The resulting energy head loss curve is conservative, since the true downstream velocity head is higher than that computed from average velocity.

The discharge rating calibration was performed for flow between about 2 and 13 cfs. At slightly above 12-cfs discharge the flow became unsteady. Discharge calibration data taken above this value were not used in determining the calibration curve. The data points were plotted, and a smooth curve (fit by eye) was drawn through them. Discharge values for heads between 0.10 foot (0.030 meter) and 0.88 foot (0.268 meter) at 0.02-foot increments were read from the curve and recorded in a table, which has been included in the design drawing, Figure 6.

After the design specifications were issued, mathematical methods were used to fit equations to the data. The best equation was then used to define the calibration curve, which yields discharge values slightly different from those reported in the table. Within the calibration data range, the differences are less than 2 percent.

A discussion of the mathematical procedures and results is presented in the next section, "Equation Fit to Discharge Calibration Data."

The two formulas suggested in the USBR Water Measurement Manual[2] for standard suppressed rectangular weirs are the Francis and the Kindsvater-Carter formulas. The Francis formula is:

$$Q = 3.33\,L\,[(H + h_v)^{3/2} - h_v^{3/2}] \qquad (1)$$

where

Q	=	discharge in cubic feet per second
L	=	length of weir crest in feet
H	=	head on weir crest in feet
h_v	=	head in feet due to average approach velocity

The Kindsvater-Carter formula is:

$$Q = C_e L_e (H_e)^{3/2} \qquad (2)$$

where, for a suppressed weir

C_e	=	3.22 + 0.40H/P
L_e	=	L − 0.003
H_e	=	H + 0.003

In the above, P is the height of the weir in feet about the floor of the approach channel, while the definition of the other terms is the same as for the Francis formula.

For the range of heads tested in the laboratory studies, the measured discharge was always higher than that computed by either of the above equations. For the maximum rated head, H = 0.88 foot, the measured discharge was 12.4 cfs, the Francis formula predicts 11.17 cfs, and the Kindsvater-Carter formula, 11.13 cfs. The measured discharge was thus about 11 percent higher than the average of the two computed values. A smaller difference existed for lower heads.

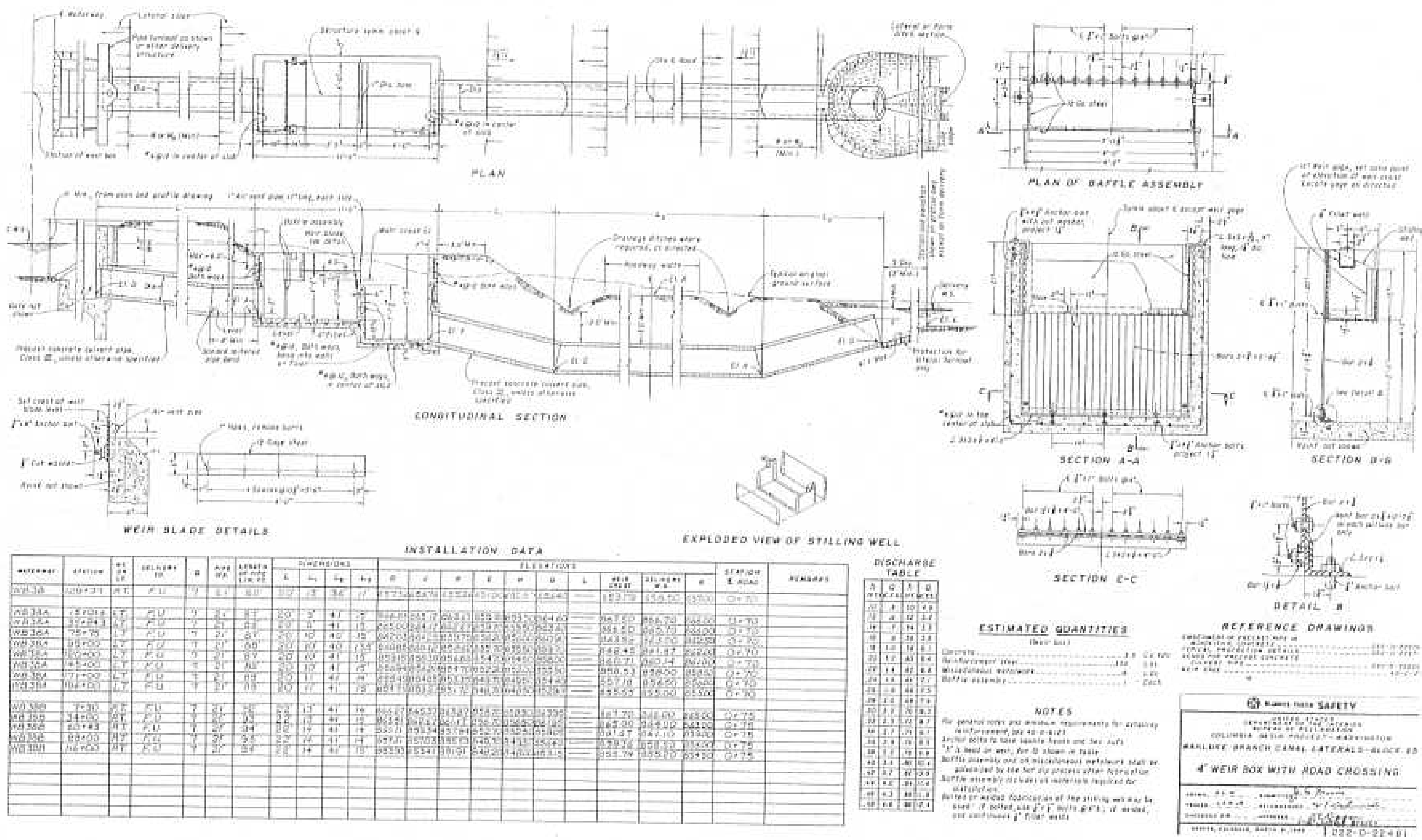


Figure 6. Specifications drawing of weir box for field use.

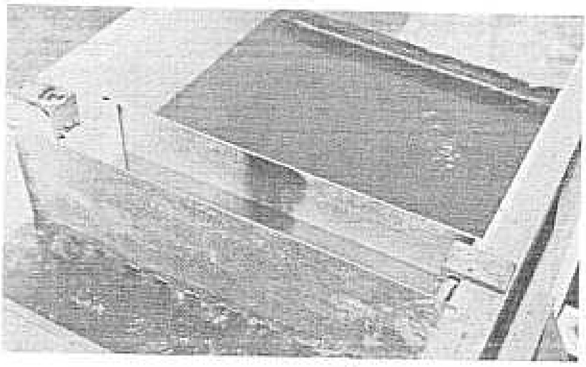


Figure 7. 5-cfs flow through weir box with recommended baffles. Photo PX-D-71977

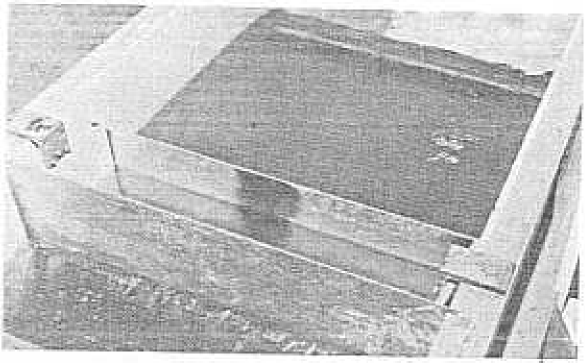


Figure 8. 7-cfs flow through weir box with recommended baffles. Photo PX-D-71978

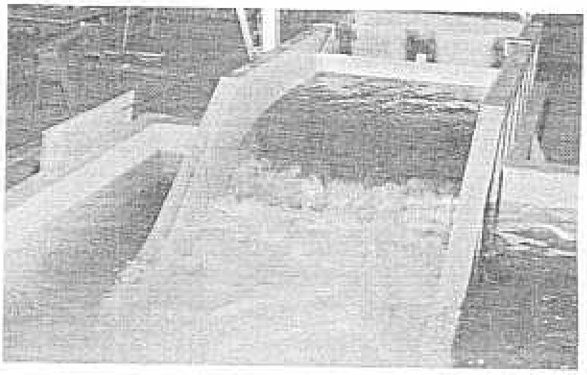
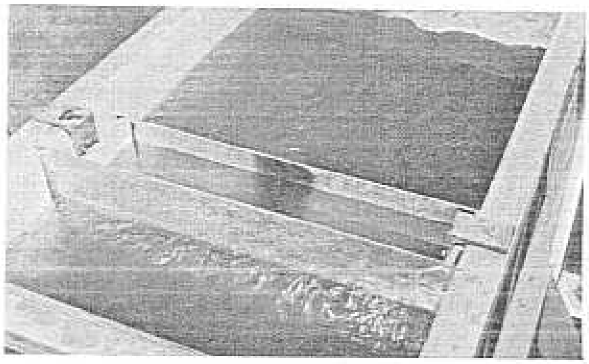


Figure 9. 10-cfs flow through weir box with recommended baffles. Photos PX-D-71982 and PX-D-71981



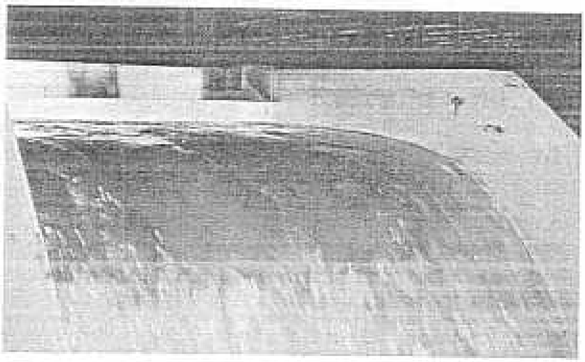


Figure 10. 13-cfs flow through weir box with recommended baffles. Photos PX-D-71979 and PX-D-71980

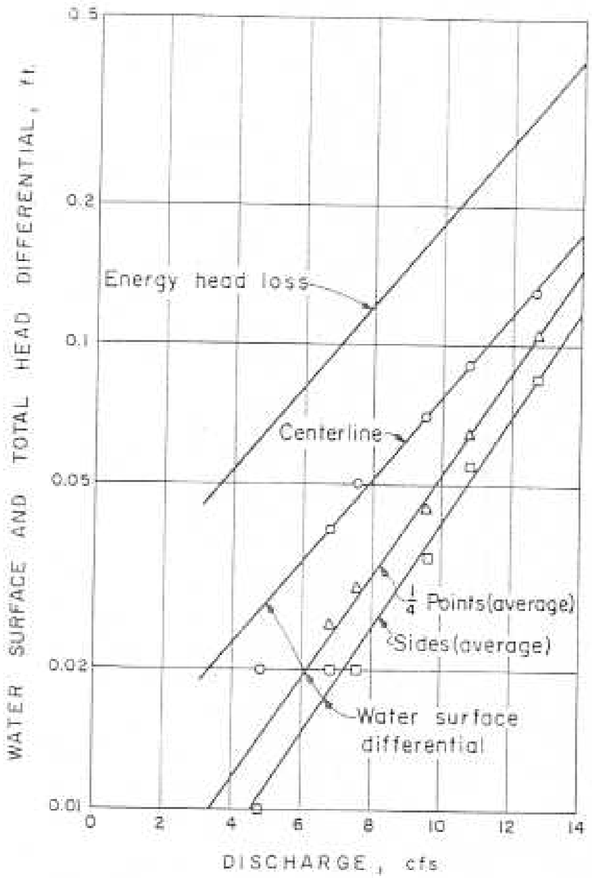


Figure 11. Water surface drop and total head loss across baffles and weir gage stilling well.

The discharge coefficient apparently is not constant over the range of heads tested. It is always higher than the discharge coefficient for standard suppressed weirs. A major reason for the coefficient being higher is the manner of measuring the head.

The weir gage stilling box, having been placed below the water surface in the approach flow, forms an obstruction to the flow. The water level measured in the well is the water surface immediately downstream of the box. The water surface level some distance downstream, however, was slightly higher (Figure 10). A staff gage placed along the wall, located 2 feet upstream of the weir, would also have indicated a higher head, and hence a lower discharge coefficient.

It is interesting to note that the level in the hook gage stilling well, which was connected to the piezometer in the floor of the box, was indeed higher than the weir gage stilling box level. Had it been used as the head measuring device, the resulting discharge coefficients would have been lower. The discharge computed from the standard equations would, however, still be as much as 8 percent higher as the measured discharge. The hook gage stilling well level was used as a check only, and no discharge rating has been attempted with its head measurements.

Other factors which could have produced higher discharge coefficients were the generally high and irregular approach velocities, and the amount of aeration of the weir overflow nappe. The cross section velocity measurements indicate that a disproportionate amount of the flow was concentrated near the water surface. The velocity was also higher under the weir gage stilling box than would exist in the approach flow of a standard installation. Thus the head due to the velocity of approach could have been considerably higher than the head based on the average velocity as used in the Francis equation.

Although the 1-inch pipe aerated the nappe, the flow of air might have been insufficient, resulting in a lower than atmospheric pressure under the nappe. This would increase the discharge above that predicted by the standard equations, which assume fully aerated nappes. No attempt was made to measure the rate of air entering below the nappe, or to check the adequacy of the 1-inch pipe size by installing larger aeration pipes.

The discharge rating was performed with the tailbox water surface level 0.1 to 0.2 foot lower than the weir crest. Impact of the falling water caused the water surface immediately below the crest to be much lower than the average tailwater level (Figure 9).

Lowering the tailwater further did not affect the rating. Raising the tailwater to the crest elevation increased the discharge by about 1 percent. Rating the weir for submergence above the crest was not attempted.

Equation Fit to Discharge Calibration Data

The lack of agreement between the discharge coefficients computed from the laboratory data and those used in generally accepted standard formulas suggested that a procedure similar to that used by Kindsvater and Carter[4] could be employed to compensate not only for fluid-property effects, but also for the effects of irregular approach flow.

[4]C. E. Kindsvater and R. W. Carter, "Discharge Characteristics of Rectangular Thin-Plate Weirs," Transactions, ASCE, Vol. 124, 1959, p 772.

The Kindsvater-Carter method employs the concept of "effective values" of head and width to eliminate the combined effects of several phenomena attributed to viscosity and surface tension. These are expressed in Equation 2:

$$Q = C_e L_e (H_e)^{3/2} \qquad (2)$$

$$H_e = H + K_h$$

$$L_e = L + K_\ell$$

A coefficient of discharge that is independent of the size of the weir or the magnitude of the head is used, and appears in Equation 2 in the form of a straight line:

$$C_e = A + B(H/P) \qquad (3)$$

In the case of a fully suppressed weir, C_e is independent of the weir width, and is a function of H/P alone. The term B(H/P) also accounts for the absence of a separate "velocity of approach" term such as required in the Francis formula (Equation 1).

The quantities K_h and K_ℓ were evaluated from experimental data by a trial computation procedure. The values of K_h and K_ℓ are determined by successive approximations as the quantities which will cause values of C_e versus H/P to plot as a straight line, and consequently define the values of A and B. Kindsvater and Carter determined the values:

$$K_h = +0.003$$

$$K_\ell = -0.003$$

$$A = 3.22$$

$$B = 0.40$$

from their own data taken with ideal approach channel conditions and carefully controlled geometry. Data from other classical weir experiments were subjected to the same procedure. The resulting constants K_h, K_ℓ, A, and B were only slightly different from the Kindsvater and Carter constants. In most cases the differences could be attributed to approach flow conditions and sharpness or rounding of the weir blade.

The Kindsvater-Carter method was applied to the weir box turnout suppressed weir data. For various values of K_h and K_ℓ the values of A and B in Equation 3 were determined by the least squares method, using a digital computer. Small values of K_h and K_ℓ ($< .010$) resulted in poor correlation coefficients, revealing considerable curvature remaining in the C_e versus H/P plot. For a wide suppressed weir the value of K_ℓ should in this case, be the same as the Kindsvater-Carter value of −0.003. Thus, it was assumed that for larger values of K_h the effect of a small K_ℓ would be negligible. K_ℓ could therefore be assumed to equal zero (i.e., $L_e = L$). For this reason, only values of K_h were substituted in subsequent trials to determine the straight line equation for C_e. After additional trials it became apparent that the best straight line fit would occur at large values of K_h ($> .100$).

Equations other than straight lines were also applied to define C_e. For lower values of K_h, a hyperbola provided the best fit. The best hyperbolic equation fit was obtained for $K_h = 0.042$, a rather high value.

The application of the Kindsvater-Carter method did not yield acceptable results. Therefore, some other method of fitting an equation to the discharge rating seemed desirable.

It was pointed out earlier that the velocity at the section where the head was measured was higher than would exist in the approach flow to the weir under ideal free surface flow conditions. The existence of the higher water surface downstream also suggested that the true velocity head at the head measuring section was considerably higher than the velocity head based on the average velocity at the same section. The true velocity head probably is only slightly lower than that existing in the constant-area section just upstream at the baffles, where the velocity head is approximately proportional to the square of the discharge. When applying the Kindsvater-Carter method to define the discharge equations, the portion of the actual velocity head that is in excess of the "ideal" must be included in K_h. The value of K_h used in the equation, however, remains constant for all discharges, while it has been shown above that the velocity head varies approximately as the square of the discharge. A constant value of K_h cannot, then, be used if a straight line equation for C_e is desired. If a convenient method of defining the excess velocity head in terms of some known quantity could be found, then a variable "effective head" could be used in combination with the Kindsvater-Carter method to define the discharge equation.

Derivation of an equation employing both a variable effective head and the Kindsvater-Carter method is presented below.

The head that would be measured with ideal approach conditions will be designated by H_T. The Kindsvater-Carter formula will be considered to be representative of ideal conditions, and, therefore, for a given Q the value of H_T will be obtained from Equation 2. The effective head, H_e would then be defined as

$$H_e = H_T + K_h = H_T + 0.003$$

In the weir box turnout model, the measured head is always less than the ideal head. Let the difference be designated by Δh,

or

$$\Delta h = H_T - H \qquad (4)$$

where H is the measured head. It will be assumed that head loss between the measuring section and weir is the same as in the ideal case and that Δh can be accounted for entirely by the excess velocity head.

In order to relate the excess velocity head to the measured head, H, it will be assumed that the excess velocity head is proportional to the velocity head through the baffles, or

$$\Delta h = K_v H_v \qquad (5)$$

where H_v = velocity head through the baffles

and K_v = proportion of H_v remaining in the form of velocity head at the measuring section

Now

$$H_v = \frac{(Q/A_b)^2}{2g}$$

where

Q = measured discharge
A_b = opening area through baffles = constant
2g = constant

Then

$$\Delta h = K_v \frac{Q^2}{2gA_b{}^2} \qquad (6)$$

Substituting $Q = C_e L_e H_e{}^{3/2}$,

$$\Delta h = K_v \frac{(C_e L_e)^2 H_e{}^3}{2gA_b{}^2} \qquad (7)$$

In the discharge range of the weir box turnout, Δh is small compared to H_e, and therefore an approximation could be used for Δh.

If we note that

$$K_v = \text{constant},$$

$$C_e \cong \text{constant, and}$$

$$H_e \cong H,$$

Equation 7 can be approximated by

$$\Delta h = KH^3 \qquad (8)$$

The value of K can be computed by substituting into Equation 8 the value of Δh determined from Equation 4. If it can be shown that K remains nearly constant for the total range of the laboratory calibration data, then the stated objective of relating Δh to some known quantity will have been met. The average value of K could then be used in determining $H_T = H + KH^3$ for use in the modified Kindsvater-Carter equation

$$Q = C_e L_e \left[H_T + K_h\right]^{3/2} \qquad (9)$$

where

$$C_e = 3.22 + 0.40\,(H_T/P) \qquad (10)$$

Table 1 lists the laboratory data for H and Q, the values of H_T computed from Equations 9 and 10 (with K_h = 0.003), and computed values of K.

Table 1

Laboratory data		Computed	
Q cfs	H ft	H_T ft	K $1/ft^2$
2.134	0.29	0.296	0.246
3.395	.395	.403	.130
4.361	.465	.476	.109
4.373	.47	.477	.067
4.415	.47	.480	.096
5.245	.525	.537	.083
5.344	.53	.544	.094
5.452	.54	.552	.076
6.795	.62	.637	.071
7.133	.635	.658	.090
7.146	.635	.658	.090
7.862	.677	.701	.077
7.915	.68	.704	.076
8.971	.735	.764	.073
9.492	.755	.793	.088
9.482	.755	.793	.088
10.123	.787	.827	.082
11.036	.817	.875	.106
11.03	.817	.875	.106
11.279	.833	.888	.095
12.215	.87	.935	.099
12.44	.877	.946	.102
12.42	.877	.945	.101

The average value for K (discarding the first value) is

$$K = .091$$

The value of K is quite sensitive to small changes in the measured value of H. The recorded data for H cannot be considered to be more accurate than plus or minus 0.01 foot. With the exception of the first two values at low heads, the computed values of K therefore appear to meet the condition that K must be nearly constant if Equation 9 is to be valid. Rounding the average value of K = 0.09 would introduce very small change in the total effective head, H_e and therefore Equations 9 and 10 can be expressed as

$$Q = C_e L_e \left[H + 0.09H^3 + 0.003\right]^{3/2} \qquad (11)$$

where

$$C_e = 3.22 + 0.40 \left[(H + 0.09H^3)/P\right] \qquad (12)$$

Laboratory measurements of Q and the values of Q computed from Equations 11 and 12 compare reasonably well. The data and results have been tabulated in Table 2. The difference in all cases is less than 2 percent, and in about two-thirds of the cases, less than 1 percent. Equations 11 and 12 can therefore be considered to be a satisfactory representation of the discharge rating.

Table 2

Laboratory data		Computed		
Q_m cfs	H ft	H_T ft	Q_c cfs	$\frac{Q_m - Q_c}{Q_c}$ percent
2.134	0.29	0.292	2.093	1.9
3.395	.395	.401	3.363	1.0
4.361	.465	.474	4.337	0.5
4.373	.47	.479	4.411	−0.9
4.415	.47	.479	4.411	0.1
5.245	.525	.538	5.254	−0.2
5.344	.53	.543	5.334	0.2
5.452	.54	.554	5.495	−0.8
6.795	.62	.641	6.864	−1.0
7.133	.635	.658	7.136	0.0
7.146	.635	.658	7.136	0.1
7.862	.677	.705	7.926	−0.8
7.915	.68	.708	7.984	−0.9
8.917	.735	.771	9.084	−1.2
9.492	.755	.794	9.502	−0.1
9.482	.755	.794	9.502	−0.2
10.123	.787	.831	10.192	−0.7
11.036	.817	.866	10.862	1.6
11.039	.817	.866	10.862	1.5
11.289	.833	.885	11.229	0.4
12.215	.87	.929	12.103	0.9
12.44	.877	.938	12.272	1.4
12.42	.877	.938	12.272	1.2

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Bureau of Reclamation

CONVERSION FACTORS—BRITISH TO METRIC UNITS OF MEASUREMENT

The following conversion factors adopted by the Bureau of Reclamation are those published by the American Society for Testing and Materials (ASTM Metric Practice Guide, E 380-68) except that additional factors (*) commonly used in the Bureau have been added. Further discussion of definitions of quantities and units is given in the ASTM Metric Practice Guide.

The metric units and conversion factors adopted by the ASTM are based on the "International System of Units" (designated SI for Systeme International d'Unites), fixed by the International Committee for Weights and Measures; this system is also known as the Giorgi or MKSA (meter-kilogram (mass)-second-ampere) system. This system has been adopted by the International Organization for Standardization in ISO Recommendation R-31.

The metric technical unit of force is the kilogram-force; this is the force which, when applied to a body having a mass of 1 kg, gives it an acceleration of 9.80665 m/sec/sec, the standard acceleration of free fall toward the earth's center for sea level at 45 deg latitude. The metric unit of force in SI units is the newton (N), which is defined as that force which, when applied to a body having a mass of 1 kg, gives it an acceleration of 1 m/sec/sec. These units must be distinguished from the (inconstant) local weight of a body having a mass of 1 kg, that is, the weight of a body is that force with which a body is attracted to the earth and is equal to the mass of a body multiplied by the acceleration due to gravity. However, because it is general practice to use "pound" rather than the technically correct term "pound-force," the term "kilogram" (or derived mass unit) has been used in this guide instead of "kilogram-force" in expressing the conversion factors for forces. The newton unit of force will find increasing use, and is essential in SI units.

Where approximate or nominal English units are used to express a value or range of values, the converted metric units in parentheses are also approximate or nominal. Where precise English units are used, the converted metric units are expressed as equally significant values.

Table I

QUANTITIES AND UNITS OF SPACE

Multiply	By	To obtain
	LENGTH	
Mil	25.4 (exactly)	Micron
Inches	25.4 (exactly)	Millimeters
Inches	2.54 (exactly)*	Centimeters
Feet	30.48 (exactly)	Centimeters
Feet	0.3048 (exactly)*	Meters
Feet	0.0003048 (exactly)*	Kilometers
Yards	0.9144 (exactly)	Meters
Miles (statute)	1,609.344 (exactly)*	Meters
Miles	1.609344 (exactly)	Kilometers
	AREA	
Square inches	6.4516 (exactly)	Square centimeters
Square feet	*929.03	Square centimeters
Square feet	0.092903	Square meters
Square yards	0.836127	Square meters
Acres	*0.40469	Hectares
Acres	*4,046.9	Square meters
Acres	*0.0040469	Square kilometers
Square miles	2.58999	Square kilometers
	VOLUME	
Cubic inches	16.3871	Cubic centimeters
Cubic feet	0.0283168	Cubic meters
Cubic yards	0.764555	Cubic meters
	CAPACITY	
Fluid ounces (U.S.)	29.5737	Cubic centimeters
Fluid ounces (U.S.)	29.5729	Milliliters
Liquid pints (U.S.)	0.473179	Cubic decimeters
Liquid pints (U.S.)	0.473166	Liters
Quarts (U.S.)	*946.358	Cubic centimeters
Quarts (U.S.)	*0.946331	Liters
Gallons (U.S.)	*3,785.43	Cubic centimeters
Gallons (U.S.)	3.78543	Cubic decimeters
Gallons (U.S.)	3.78533	Liters
Gallons (U.S.)	*0.00378543	Cubic meters
Gallons (U.K.)	4.54609	Cubic decimeters
Gallons (U.K.)	4.54596	Liters
Cubic feet	28.3160	Liters
Cubic yards	*764.55	Liters
Acre-feet	*1,233.5	Cubic meters
Acre-feet	*1,233,500	Liters

Table II

QUANTITIES AND UNITS OF MECHANICS

Multiply	By	To obtain
MASS		
Grains (1/7,000 lb)	64.79891 (exactly)	Milligrams
Troy ounces (480 grains)	31.1035	Grams
Ounces (avdp)	28.3495	Grams
Pounds (avdp)	0.45359237 (exactly)	Kilograms
Short tons (2,000 lb)	907.185	Kilograms
Short tons (2,000 lb)	0.907185	Metric tons
Long tons (2,240 lb)	1,016.05	Kilograms
FORCE/AREA		
Pounds per square inch	0.070307	Kilograms per square centimeter
Pounds per square inch	0.689476	Newtons per square centimeter
Pounds per square foot	4.88243	Kilograms per square meter
Pounds per square foot	47.8803	Newtons per square meter
MASS/VOLUME (DENSITY)		
Ounces per cubic inch	1.72999	Grams per cubic centimeter
Pounds per cubic foot	16.0185	Kilograms per cubic meter
Pounds per cubic foot	0.0160185	Grams per cubic centimeter
Tons (long) per cubic yard	1.32894	Grams per cubic centimeter
MASS/CAPACITY		
Ounces per gallon (U.S.)	7.4893	Grams per liter
Ounces per gallon (U.K.)	6.2362	Grams per liter
Pounds per gallon (U.S.)	119.829	Grams per liter
Pounds per gallon (U.K.)	99.779	Grams per liter
BENDING MOMENT OR TORQUE		
Inch-pounds	0.011521	Meter-kilograms
Inch-pounds	1.12985×10^6	Centimeter-dynes
Foot-pounds	0.138255	Meter-kilograms
Foot-pounds	1.35582×10^7	Centimeter-dynes
Foot-pounds per inch	5.4431	Centimeter-kilograms per centimeter
Ounce-inches	72.008	Gram-centimeters
VELOCITY		
Feet per second	30.48 (exactly)	Centimeters per second
Feet per second	0.3048 (exactly)*	Meters per second
Feet per year	*0.965873×10^{-6}	Centimeters per second
Miles per hour	1.609344 (exactly)	Kilometers per hour
Miles per hour	0.44704 (exactly)	Meters per second
ACCELERATION*		
Feet per second^2	*0.3048	Meters per second^2
FLOW		
Cubic feet per second (second-feet)	*0.028317	Cubic meters per second
Cubic feet per minute	0.4719	Liters per second
Gallons (U.S.) per minute	0.06309	Liters per second
FORCE*		
Pounds	*0.453592	Kilograms
Pounds	*4.4482	Newtons
Pounds	*4.4482×10^5	Dynes

Table II—Continued

Multiply	By	To obtain
WORK AND ENERGY*		
British thermal units (Btu)	*0.252	Kilogram calories
British thermal units (Btu)	1,055.06	Joules
Btu per pound	2.326 (exactly)	Joules per gram
Foot-pounds	*1.35582	Joules
POWER		
Horsepower	745.700	Watts
Btu per hour	0.293071	Watts
Foot-pounds per second	1.35582	Watts
HEAT TRANSFER		
Btu in./hr ft^2 degree F (k, thermal conductivity)	1.442	Milliwatts/cm degree C
Btu in./hr ft^2 degree F (k, thermal conductivity)	0.1240	Kg cal/hr m degree C
Btu ft/hr ft^2 degree F	*1.4880	Kg cal m/hr m^2 degree C
Btu/hr ft^2 degree F (C, thermal conductance)	0.568	Milliwatts/cm^2 degree C
Btu/hr ft^2 degree F (C, thermal conductance)	4.882	Kg cal/hr m^2 degree C
Degree F hr ft^2/Btu (R, thermal resistance)	1.761	Degree C cm^2/milliwatt
Btu/lb degree F (c, heat capacity)	4.1868	J/g degree C
Btu/lb degree F	*1.000	Cal/gram degree C
Ft^2/hr (thermal diffusivity)	0.2581	Cm^2/sec
Ft^2/hr (thermal diffusivity)	*0.09290	M^2/hr
WATER VAPOR TRANSMISSION		
Grains/hr ft^2 (water vapor transmission)	16.7	Grams/24 hr m^2
Perms (permeance)	0.659	Metric perms
Perm-inches (permeability)	1.67	Metric perm-centimeters

Table III

OTHER QUANTITIES AND UNITS

Multiply	By	To obtain
Cubic feet per square foot per day (seepage)	*304.8	Liters per square meter per day
Pound-seconds per square foot (viscosity)	*4.8824	Kilogram second per square meter
Square feet per second (viscosity)	*0.092903	Square meters per second
Fahrenheit degrees (change)*	5/9 exactly	Celsius or Kelvin degrees (change)*
Volts per mil	0.03937	Kilovolts per millimeter
Lumens per square foot (foot-candles)	10.764	Lumens per square meter
Ohm-circular mils per foot	*0.001662	Ohm-square millimeters per meter
Millicuries per cubic foot	*35.3147	Millicuries per cubic meter
Milliamps per square foot	*10.7639	Milliamps per square meter
Gallons per square yard	*4.527219	Liters per square meter
Pounds per inch	*0.17858	Kilograms per centimeter

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