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Experimental Coefficient of Discharge for Leaky Woody Dams in Clear Water Conditions

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ABSTRACT

Natural flood management (NFM) uses natural features and processes to manage flood risk. Although natural processes in river flow are well known, their use to manage floods has been only deeply analyzed since recent years, and the hydraulic behavior and performance of some measures is not yet fully understood. Leaky woody dams (LWD) are one example of the application of NFM, which is very complex to understand, and many uncertainties are still unresolved. In practical applications, the effect of LWD is modeled with different assumptions (Manning's n, geometry changes, porosity, etc.), but none of them represent the real physics of the problem. This paper presents novel lab experiments that attempt to simulate LWD in a river channel. The experiments were developed in a straight research flume. The performance of the LWD in terms of outflow capacity and the effect in terms of increase in water levels upstream and velocities downstream have been analyzed. The orifice + weir model has been proposed as the more realistic model to simulate the flow through LWD, and empirical coefficients of discharge for applications in analytical methods and in numerical models have been obtained. The results help to understand the hydraulic behavior of LWD, and the coefficients of discharge obtained can be useful to reduce uncertainties in numerical modeling for practical applications.

1 | Introduction

Flooding is the most devastating natural disaster in the UK, costing the nation around 2.2 billion pounds each year in damages (Evans et al. 2004; Sayers et al. 2020). The relocation of entire communities, properties, and financial activities can be very expensive and unreasonable, making flood defence measures the necessary solution in flood-prone areas. However, traditional flood defences are becoming less effective due to urbanization development and climate change, increasing the number and intensity of flood events. Traditional practices improve channel efficiency by increasing flow velocity in streams (Gippel 1995) but lead to significant degradation of river ecology, biodiversity, and morphological variability (Howe and White 2003). The idea of introducing more sustainable and nature-based flood management methods has been growing over the past years (Pitt 2008; Dadson et al. 2017; World Bank 2017). These nature-based solutions (NbS) are more cost-effective and environmentally friendly than traditional flood defences, making them more sustainable (EC 2014; Roca et al. 2017).

The term Natural Flood Management (NFM) refers to the flood management techniques that use natural processes to restore natural flow regimes, which typically means extending the time taken for peak discharge to be reached in rivers by bringing down and slowing the runoff. These two effects help to reduce the frequency and severity of flooding downstream by retaining water and changing the flow profile of a catchment (Figure 1). NFM applies a series of underlying principles: promotion of interception, infiltration, and retention of runoff. This is achieved typically using "soft" engineering solutions, such as Leaky Woody Dams (LWD), wetland and pond creation, field bunds, riparian buffer strips, and reconnection of rivers with flood plains. NFM attempts to consider the whole of a catchment area rather than traditional local flood protection. Such schemes tend to be located in upland rural catchments where funding for larger scale more traditional flood control measures does not satisfy required cost-benefit ratios due to high frequency flooding but limited damages (Wilkinson et al. 2014). The idea of decentralized flood management is also important. Rather than implementing a small number of large capital schemes situated in lower parts of a catchment, a multitude of small "interventions" are situated, typically, toward the upper end of catchment areas in small tributary catchments (Reinhardt et al. 2011; Ramsbottom et al. 2019). These small interventions can also be coupled with large scale catchment land use changes to increase catchment roughness (normally through afforestation), as well as re-naturalization of water courses and floodplains (Gilvear and Casas 2008). Over the last 20 years, NFM has gained traction within the field of Flood Risk Management, with NFM measures being included as part of schemes up and down the UK (Bannister et al. 2005; Nisbet, Thomas, and Roe 2015; Stroud District Council and Environment Agency 2018; Lavers and Charlesworth 2018). However, NFM techniques require additional research to prove their benefits (Gippel 1995; Ramsbottom et al. 2019).

One of the nature-based techniques that have been widely used is installing wood dams or barriers to support in channel attenuation. These structures are known as "leaky dams" or "leaky woody dams" (LWD), and they can minimize the risk of a flood as they delay downstream peak flows by rising of water level and pushing it out of the channel and onto the floodplain, supporting out-of-bank spillover (Wilt and Corwin 1989). Fundamentally, the role of a Leaky Woody Dam (LWD) is to slow the flow by partially blocking the water stream and increasing the resistance of the water to flow. The aim of slowing the flow is to hold the rainwater upstream away from the flood risk area during the peak of any flood event and then desynchronize the flash floods from the peak of the flood event in urbanized and affected areas (Odoni and Lane 2010). The LWD usually run in series, with each storing water temporarily. When the rain reduces and river levels drop, the stored water is then released through the leaks in a naturally controlled manner. This could reduce the impact of rainfall into the downstream catchment when the river levels are high (van Leeuwen et al. 2024).

LWD should be specially designed to not interfere with the normal river flow and the fish that live in it (Figure 2). Their design can vary in complexity, ranging from some simple posts driven into the banks and bed to catch and accumulate

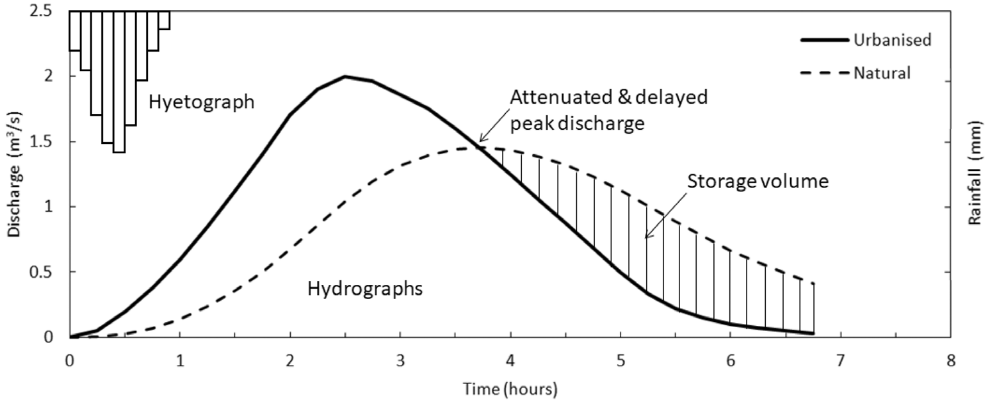

FIGURE 1 | Natural versus urbanized catchment hydrographs, adapted from Woods-Ballard et al. (2007). The natural hydrograph reduces the peak discharge and hence the frequency and severity of flooding downstream.

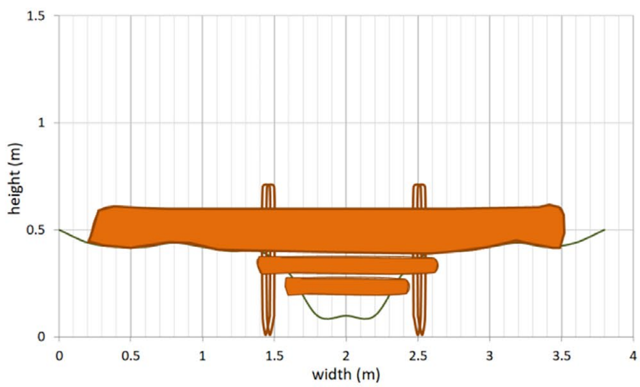

FIGURE 2 | Group of leaky woody dams in Ruislip Woods (left). Standard basic "design" of a LWD (right).

natural woody debris traveling down the channel, to multiple longer lengths positioned across the channel and "floodplain" and dug into banks and pinned with wooden or metal reinforcement (Forbes et al. 2015; Woodland Trust 2016; Yorkshire Dales River Trust guidance 2021; CIRIA 2022). There are risks that wood from dams can become mobilized and can increase the risk of blockages causing potential flooding at unintended locations or damage to structures such as bridges or weirs. Other risks related to LWD are local scour and geomorphology changes in the streams (Wolstenholme et al. 2024; Mcparland 2021; Furnues 2023). There is a need to understand the individual role of LWD and to what extent they assist other flood management processes and techniques. For this reason, modeling LWD is appropriate so that their individual effect can be isolated. Several hydrological models have attempted to assess the performance of groups of LWD in different UK catchments (Odoni and Lane 2010; Kitts 2010; Thomas and Nisbet 2012; Dixon 2013; Nisbet, Roe, et al. 2015; Nicholson et al. 2020; Follett and Hankin 2022). In general, flow peak reduction downstream of leaky woody dams was reported for different modeling scenarios (Senior et al. 2022). Approaches between papers vary, and they are summarized in Chapter 2. There is certainly not a "typical" method presented in the literature (Addy and Wilkinson 2019), which raises the question of what is an appropriate method for modeling LWD.

Both field and modeling studies have assessed the benefits of naturally occurring large wood in channels (Gregory et al. 1985; Braudrick and Grant 2001; Klaar et al. 2020) and beaver dams (Nyssen et al. 2011; Puttock et al. 2021). However, artificially deployed LWD is unlikely to have the same shape, density, and behavior as natural wood accumulations or beaver dams, and they also tend to be secured in place, unlike natural in-channel wood (Muhawenimana et al. 2021). Recent field studies (Nisbet et al. 2011; Nicholson et al. 2012; Short et al. 2019; Slow the Flow 2021; Thames21 2021; Lavers et al. 2022; Muhawenimana et al. 2023) have reviewed the benefits of leaky barriers, showing that they can delay the travel time of the flood peak and demonstrating that although the initial peak flow can be reduced, successive rainfall peaks are less effective as storage becomes mobilized. Although monitoring was included in some of these projects (Norbury et al. 2021), most of them include several interventions, not only LWD, so quantifying the effect of LWD in particular is difficult, and more monitoring and analysis on individual types of NFM interventions have to be done.

In contrast to the number of field studies on LWD, only a few experimental tests have been found in the literature. Some initial laboratory studies of woody dams have assessed the effects of single woody elements (Young 1991) and how their position in the river channel influences the water levels. One of the first researchers who attempted to model the flow through a leaky dam with a sluice gate in a flume was Leakey et al. (2020, 2022). The laboratory model only simulated flow under the gate and overtopping in combination with underneath flow. But the sluice gate was not circular, and no intermediate orifices were modeled. Recently, the research group at Cardiff University has carried out several experiments with round cylinders in a larger flume. Muhawenimana et al. (2021) and Follett et al. (2020 and 2021) addressed the research gaps about the backwater effects of different LWD designs and the physical characteristics that determine the extent of flood attenuation. Porous and non-porous leaky barrier designs were tested in a laboratory flume. They included the circular logs in a mesh of logs, which do not individualize the effect of each log on the coefficient of discharge. Their analysis used the porosity of the whole dam as the main parameter, rather than the shape of the logs, roughness, and coefficient of discharge.

This paper presents novel laboratory experiments for understanding and assessing the performance of LWD under ideal and natural conditions. The combined effects of pressure flow and overtopping flow are analyzed to characterize the different types of flow occurring through the LWD and their logs and to estimate different coefficients of discharge for different flow scenarios. Based on the experimental results, an empirical Coefficient of Discharge is proposed to be applied in the weir/orifice equations for estimating the flood attenuation on LWD in practical situations.

2 | Hydraulics of Leaky Woody Dams. 1D Equations

When considering the existing equations and model approaches to quantify the effects of LWD, several approaches have been proposed in the literature:

- To use a flow resistance equation: typically with an increase in Manning's n friction factor (Kitts et al. 2003).
- To model the LWD as hydraulic structures, where the flow rate is calculated with a weir/orifice equation (Metcalfe et al. 2017).
- To model LWD as changes in geometry, where the cross-sectional area is decreased, by raising the bed level or narrowing the banks (Thomas and Nisbet 2012; Valverde 2013).
- The porosity-wall model (Thames21 2021) or the canopy drag model (Follett et al. 2020, 2021).

These five traditional models of flow discharge (open channel flow, orifice flow, weir flow, flow under a narrow section, and porous flow) have been applied to a rectangular cross-section, 0.3 m width, with horizontal bed, to simulate the theoretical effect of a LWD. The results are graphically represented in Figure 3. Each model of flow is different and gives very different rating curves in a rectangular section with horizontal bed. As the flow through a LWD is a combination of pressure flow under the logs and free overtopping in some cases and when the water reaches the top log, this paper proposes the combination of orifice and weir flow as the more realistic to flow in LWD. In some cases where debris block the gap between logs, this method could be not the best option. The values of Manning's n, width of LWD section and porosity have been calibrated to get similar values of depth for $Q = 5$ L/s. The rating curves diverge from this point with a big difference for $Q = 10$ L/s, where the depths vary from 8 cm (Manning model) to 14 cm (Orifice + Weir model), Hankin et al. (2020) divided the flow passing the LWD into only three modes (Figure 4).

The parameters which control the flow are: h—depth of the water behind the LWD, a—depth of flow under pressure, D—height of top lock over bottom, hi—depth of water over the weir, and h_0—depth of water downstream of the LWD. The dam has its bottom at height a—above the streambed and its top at height D—(Figure 4).

Mode 1: $h < a$, open channel flow (the water level is below the bottom and the dam is doing nothing).

Mode 2: $a \leq h < D$, flow under pressure (when the dam is operating normally).

Mode 3: $h > D$, overtopping flow and flow under pressure (when the dam is overspilling).

For the first mode (free flow), the Manning equation can be used (Equation 1), while for the second two modes (pressure flow and free overtopping flow), the hydraulic theory for the flow under sluice gates and over weirs can be used (e.g., Swamee 1992).

Following Hankin approach, the equations of pressure flow through an orifice (Swamee 1992; Castro-Orgaz et al. 2010; Kiczko et al. 2015 and Kubrak et al. 2020) or the flow over a curvilinear weir (Matthew 1963; Castro-Orgaz et al. 2008; Castro-Orgaz and Hager 2014; and Herrera-Granados and Kostecki 2016) are proposed here as the best option to simulate the flow in LWD (Equations 1 and 2).

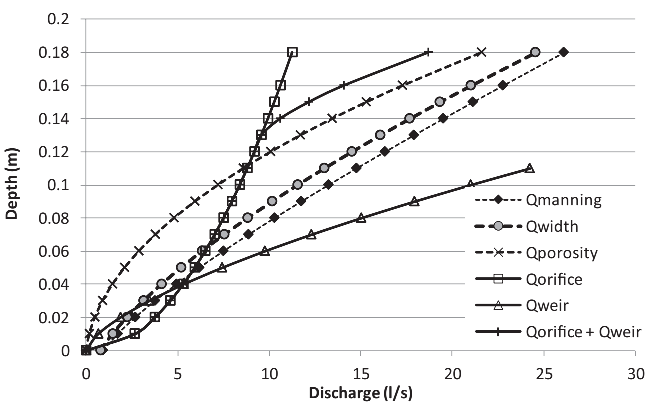

FIGURE 3 | Water depths versus discharge for different hydraulic equations applied to leaky dams. QManning means change of Manning's n in leaky dam cross sections, Qwidth means change of channel width, Qstep means change in channel elevation, Qorifice means use of orifice equation, Qweir means use of weir equation, and Qorifice+weir means use of both after the crest level is reached.

Pressure Flow under a small orifice (from Bernoulli's conservation of energy equation):

$$Q = b \cdot a \cdot \frac{1}{\left(1+K_L\right)^{1/2}} \sqrt{2g}\left(H-h_0\right)^{1/2} \tag{1}$$

$$\text{with } K_L = 0.5 \text{ as maximum theoretical value, so } C_d = \frac{1}{\left(1+K_L\right)^{1/2}} = 0.81 \text{ and } a = \text{constant}$$

Weir equation:

$$Q = b \cdot C_o \sqrt{2g}(H-D)^{3/2} \tag{2}$$

$$\text{with } C_{do} = (2/3)^{1.5} = 0.54 \text{ as maximum theoretical value.}$$

where H = head energy upstream of the LWD (usually is equal to h = depth upstream), h_0 = depth downstream of LWD, a = depth of flow under the LWD, D = height of the LWD from bed to top of logs, b = width/length of the orifice/weir, g = gravitational acceleration and C_d and C_o are coefficients of discharge for pressure flow through an orifice and for overtopping, respectively.

3 | Setup of Laboratory Experiments

These laboratory experiments are based on the collaboration between Brunel University London (BUL) and Ruislip Woods Advisory Management Group (RWAMG). The Ruislip Woods are a Natural England protected area whose catchment is contributing to the residential area close to the river Pinn at Ruislip. After some major flood events since 2000, the Environment Agency founded a flood management project in collaboration with Hillingdon County Council (Thames21 2021). The project included the construction of 50 LWD in Ruislip Woods to slow the flow and reduce the flood risk downstream of the Woods. The numerical modeling carried out showed some attenuation, but the monitoring of the LWD was not effectively implemented. The experiments carried out at BUL are based on the design of some of the LWD in Ruislip Woods (Figure 5).

Physical modeling experiments were undertaken for a LWD model (based on the one in Figure 5) in a recirculating open channel flume in the Centre of Flood Risk and Resilience at Brunel University London (Figure 6). The experiments were developed in a straight research flume, so no storage effects have been analyzed. The flume has length 5.0 m (lflume), width 0.3 m (bflume), and depth 0.5 m (hflume). The longitudinal flow is defined as the main flow in the x direction, and vertical and lateral

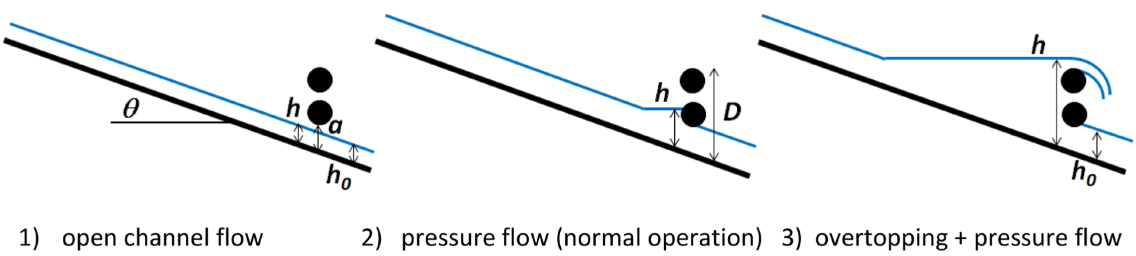

FIGURE 4 | Flow modes in a leaky woody dam, after Hankin et al. (2020).

coordinates were defined as y and z, respectively. A pump with a 20 L/s capacity recirculated the water and controlled the discharge, while a sharp crested tailgate weir located at the downstream end of the flume controls the outflow with a critical depth transition to supercritical flow downstream.

The flume comprises a horizontal longitudinal bed slope with a rectangular cross-section of width $b = 0.3$ m and variable depths depending on the flow rate. The roughness of the flume bottom and walls was experimentally calibrated, giving a Manning's n of 0.011. This value does not represent field conditions but just the roughness of the surfaces in the lab. A physical model of a leaky woody barrier was placed at $x = 2.00$ m from the downstream outlet (Figure 6). The LWD was formed with 3 logs (log 1-bottom, log 2-mid, and log 3-top) and 3 orifices (bottom, orifice 1–2, and orifice 2–3). The diameter and opening height of the orifices can be seen in Figures 7 and 8. Four LWD configurations were tested for two different materials in the logs (smooth

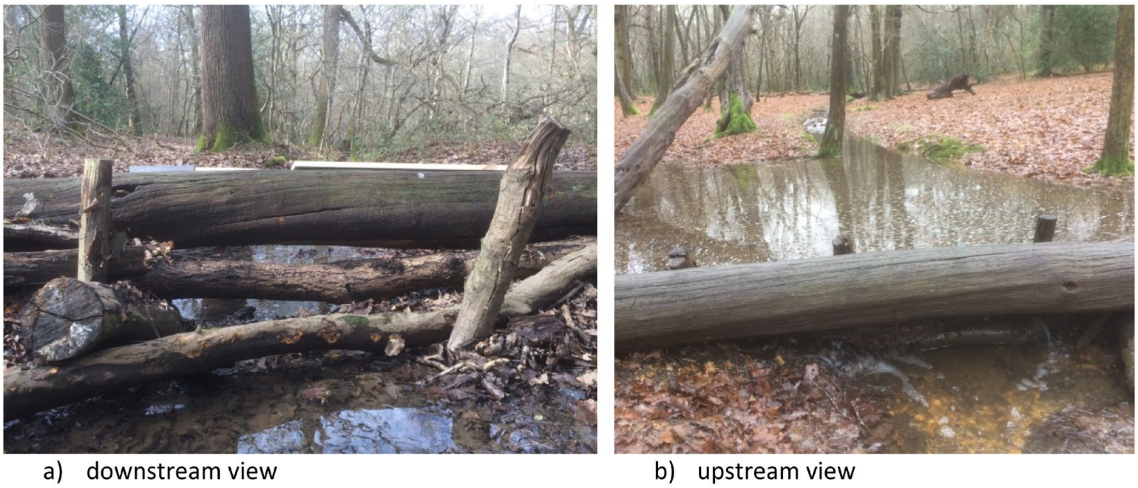

FIGURE 5 | Woody leaky dam at Ruislip Woods. Downstream and upstream views.

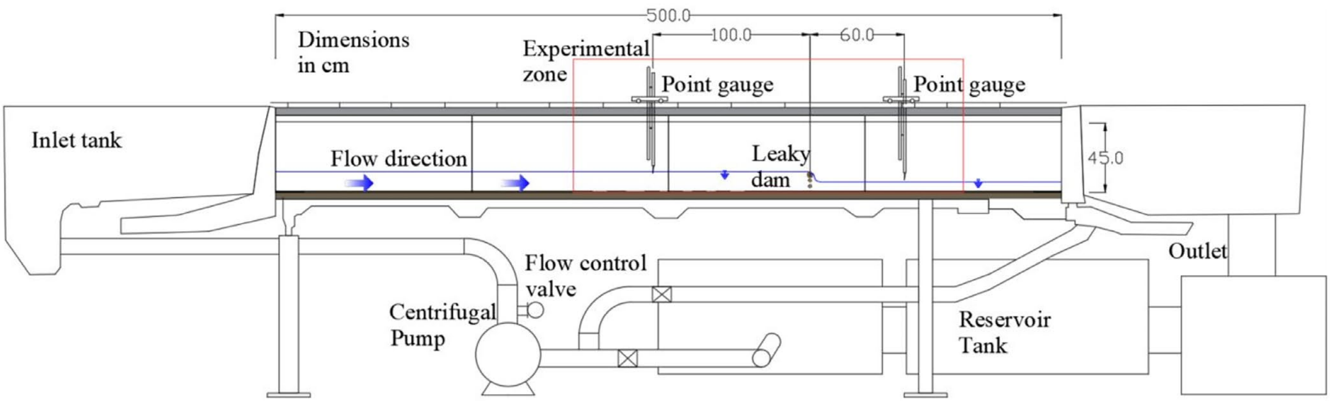

FIGURE 6 | Sketch of the experimental flume and location of the Leaky dam model.

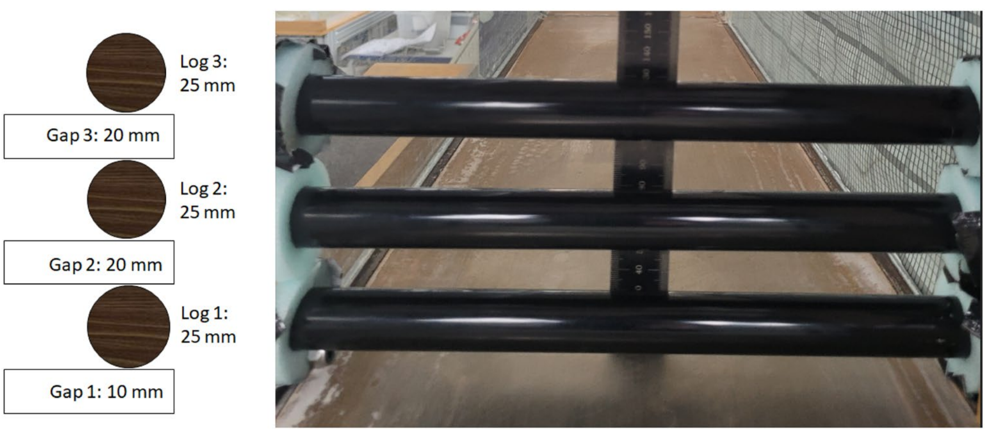

FIGURE 7 | View of the Leaky dams with smooth logs (right) and dimensions of logs and gaps (left). Smooth-U and Smooth-S tests.

FIGURE 8 | View of the Leaky dams with natural logs (right) and dimensions of logs and gaps (left). Natural-U and Natural-S.

TABLE 1 | Summary of tests for smooth/natural logs with unsubmerged/submerged flow conditions.

Type		*h* (mm)	h_0 (mm)	*Δh* (mm)	*Q* (l/s)	*Fo Froude n.*	*Re Reynolds n.*	*h/a*
Smooth- U	Smooth/circular logs unsubmerged flow	17–155	13–57	4–98	1–20	0.5–1.4	6600–60,000	1.7–4.1
Smooth- S	Smooth/circular logs submerged flow	17–149	13–118	4–31	1–16	0.4–0.5	6600–60,000	1.7–4.0
Natural- U	Natural logs unsubmerged flow	23–152	17–45	6–107	1–15	0.5–1.7	6600–50,000	1.4–3.9
Natural- S	Natural logs submerged flow	18–151	18–111	3–40	1–15	0.4–0.5	6600–50,000	1.0–3.6

and natural) and for two steady-state flow conditions (Table 1). For each of the four configurations, 15 discharges were tested from 1 to 15 L/s, the discharge that overtopped the logs about 2 cm (60 data tests in total).

Two downstream flow conditions are tested: unsubmerged and submerged flow. Unsubmerged flow refers to scenarios where the flow downstream is free, mostly fast flow (Fo > 1) and no affected by boundary condition downstream, and submerged refers to flow controlled by downstream conditions (Fo < 0.6), and all the orifices are submerged at some stage. The downstream tail water affects the flow through the LWD. Depth measurements were taken at three fixed points along the flume, two upstream of the barrier and only one downstream: h12, h13, h17.8, where the subscript denotes the distance in metres along the flume.

The boundary conditions were the discharge upstream and the critical depth downstream for the free flow experiments. In submerged experiments, a low weir was located downstream to force the subcritical flow, and measured depths for each flow were used as downstream water level. For each experiment, the steady-state flow was measured with a flow meter inserted in the inflow pipe between the pump and flume, and depths were measured with point gauges.

The physical model of the leaky barrier consists of three horizontal wood logs with a gap underneath and gaps between the logs (Figures 7 and 8, and Table 2). Initially the flow always pass under the logs (pressure flow) and once it reaches the top of each log there is flow over the top log too (overtopping flow). After that (pressure + overtopping flows), there is a point that the water reaches the log above and the flow changes from overtopping to pressure flow. Hence, there are seven possible flow conditions (operational stages) for the whole LWD (Table 3 and Table 5 in results):

TABLE 2 | Geometric parameters of the logs and gaps in the two sets of experiments.

(a) Smooth-U and -S experiments			(b) Natural-U and -S experiments		
Gap/log	Size (mm)	Total depth (mm)	Gap/log	Size (mm)	Total depth (mm)
Gap 1	10	10	Gap 1	17	17
Log 1	25	35	Log 1	25	42
Gap 2	20	55	Gap 2	14	56
Log 2	25	80	Log 2	26	82
Gap3	20	100	Gap3	8	90
Log 3	25	125	Log 3	40	130

The two first set of experiments were with smooth logs (Figure 7). They can be compared with natural logs to understand the effect of natural features (roughness, spacing and shape) in the flow. The

TABLE 3 | Flow conditions identified in a typical leaky dam tested in the lab.

No	Gap/log	Operational stages	Flow condition	Types of flow		
1	Gap1	Water under log 1	Open channel flow	Free flow		
2	Log1	Water reaches log 1	Pressure flow under log 1		Pressure flow	
3	Log1/Gap2	Water overtops log 1	Overtopping over log 1 and pressure flow under log 1		Pressure flow	Overtopping flow
4	Log2	Water reaches log2	Pressure flow under log 2 and log 1		Pressure flow	
5	Log2/Gap3	Water overtops log2	Overtopping over log 2 and pressure flow under logs 2 and 1		Pressure flow	Overtopping flow
6	Log3	Water reaches log3	Pressure flow under log 3, log 2 and log 1		Pressure flow	
7	Log3/Gap3	Water overtops log 3	Overtopping over log 3 and pressure flow under logs 3, 2 and 1		Pressure flow	Overtopping flow

main geometric features are summarized in Table 2a. The smooth logs (plastic) were uniformly placed through the depth of the flume, creating a homogeneous and ideal geometry of a leaky dam.

The second set of experiments was carried out with natural logs taken from the Ruislip Woods (Thames21 2021). Figure 8 illustrates the main features of these experiments and the natural characteristics of the logs. The main geometric features are summarized in Table 2b. The natural logs were placed trying to simulate some of the woody leaky dams built in Rusilip Woods (Figure 8), with a wider log at the top and smaller at the bottom. This model is a representation of common designs of LWD found in the literature.

4 | Experimental Results

In the first instance, only the flow under pressure is considered, ignoring the data with flow over the logs. The Coefficient of Discharge, C_d, for pressure flow under the logs is analyzed and compared with theoretical values (C_{dt}) obtained for sluice gates in lab conditions. For low flows, the bottom log (log 1) is considered isolated, but for high flows, it is assumed that the bottom log and intermediate and top logs (logs 2 and 3) act as only one gap under pressure flow (the gap between different logs is taken as a bottom gap), so the coefficients obtained for each log consider only one gap, the sum of the bottom and intermediate gaps. The equation of pressure flow under a small orifice or sluice gate (Equation 1) is used to calibrate the C_d coefficient for LWD (from Bernoulli's conservation of energy principle).

The results obtained for smooth logs both unsubmerged (free) and submerged downstream flows are represented in Figure 9. Figure 9 (left) shows the values of C_d for each combination of logs (log1, logs1-2, logs 1-2-3) under different h/a ratios. Figure 9 (right) shows the same results but for individual h/a ratios in different logs, confirming that the behavior of the pressure flow under logs 2 and 3 is the same as under only one log, log 1. In the figures, it can be identified that after a value of $h/a > 2$, the C_d follows a straight trend which is slightly higher for unsubmerged flow downstream, increasing from $C_d = 0.6$ for low h/a ratios to $C_d = 0.9$ for high h/a ratios. The values for submerged flow conditions are lower than unsubmerged tests for the same h/a ratios. Values for similar h/a have been isolated to compare the results and to understand that the trend is the same independently of the h/a ratio, as confirmed by theoretical values. It is evident from Figure 9 that the free flow downstream of the LWD has a strong influence on the C_d for the smooth logs, compared with the submerged flow, giving higher values of C_d (0.5–0.7 for the submerged cases vs. 0.5–0.9 for the free flow cases). The difference is greater for high h/a values.

The results obtained for natural logs both unsubmerged (free) and submerged downstream flows are shown in Figure 10. The results show that the pressure C_{d} for each individual log under different h/a ratios follows a similar trend to smooth logs (Figure 10). Figure 10 (right) shows the same results but for individual h/a ratios in different logs, confirming that the behavior of the pressure flow under logs 2 and 3 is the same as under only one log, log 1. In the figures, it can be identified that after a value of $h/a > 2$, the C_d follows a straight trend which is slightly higher for unsubmerged flow downstream, increasing from $C_d = 0.6$ for low h/a ratios to $C_d = 0.9$ for high h/a ratios. The values for submerged flow conditions are lower, varying from $C_d = 0.5$ to $C_d = 0.8$ for the same h/a ratios as unsubmerged tests. Values for similar h/a have been isolated to compare results, and this helps to understand that the trend is the same independently of the h/a ratio, as confirmed by theoretical values.

In both the smooth and natural logs, the value of C_d differs from the theoretical values, C_{dt}, obtained in flows under a sluice gate, as they do not trend to a constant value. More experimental work is needed in order to understand the reason for this, and obtain a more general relationship of C_d–h/a.

Comparing the smooth logs with the natural logs, it is confirmed that the pressure C_d for the smooth ones is bigger than the natural ones when the flow under the logs is free (unsubmerged),

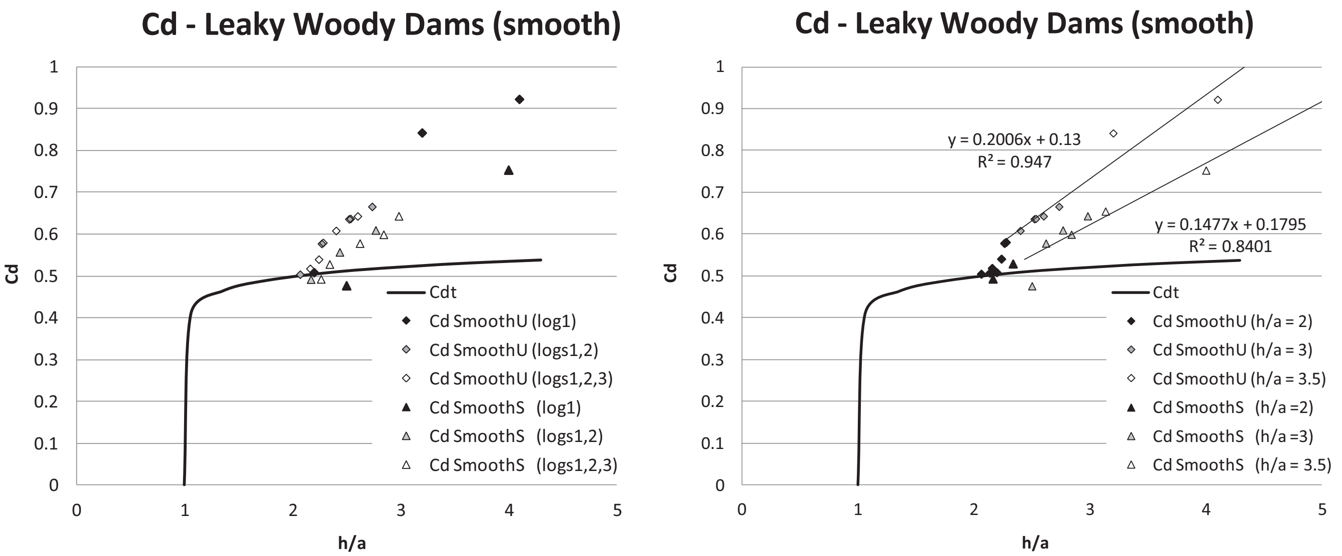

FIGURE 9 | C_d measured for pressure flow under Smooth logs. C_{dt} = theoretical value. Left: Variation for each log. Right: Variation for h/a ratio. SmoothU means unsubmerged flows and SmoothS means submerged flows. Values for similar h/a have been isolated to compare results. $h/a = 2$ means $h/a = 1.7–2.5$, $h/a = 3$ means $h/a = 2.6–3.0$, and $h/a = 3.5$ means $h/a > 3$.

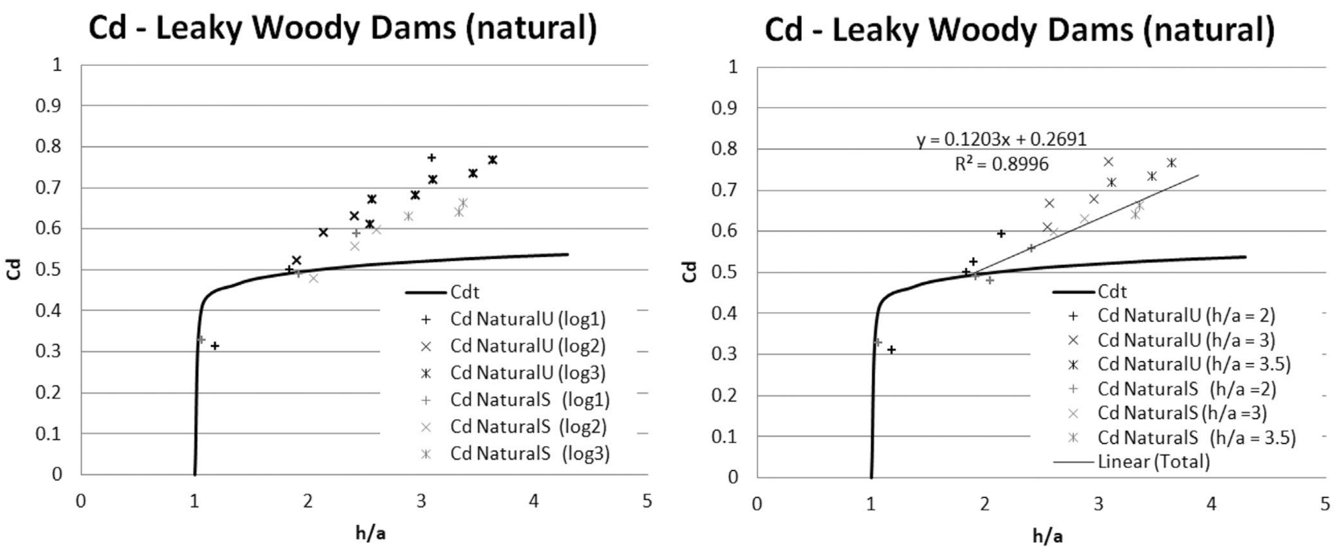

FIGURE 10 | C_d for pressure flow under Natural logs. Left variation for each log. Right variation for h/a ratio. Natural-U means unsubmerged flows and Natural-S means submerged flows. Values for similar h/a have been isolated to compare results. $h/a = 2$ means $h/a = 1.7–2.5$, $h/a = 3$ means $h/a = 2.6–3.0$, and $h/a = 3.5$ means $h/a > 3$.

Figure 11a. However, when the flow is submerged, the roughness of the logs has no influence in C_d, and only the ratio h/a is influencing the variation of C_d, almost linearly (Figure 11b). This variation is like in theoretical values obtained for free flow under sluice gates (Swamee 1992) but with a higher slope due to the round shape of the logs (Singh et al. 2025). In summary (Figure 11c), the C_d varies similarly in both smooth/natural logs in submerged conditions, showing that the effect of log roughness is almost canceled by the submergence. However, in free flow conditions (unsubmerged), the roughness of the logs has a strong effect in C_d. For the natural logs, the values of C_d only vary with the ratio h/a, but not with the ratio of submergence.

Table 4 summarizes the values of C_d estimated in the lab for each case analyzed. Table 5 shows the full set of data and numerical calculations for C_d in the case of Natural-U. Once the C_d for flow under pressure is determined, the objective is to obtain the values for the Coefficients of Discharge in overtopping C_o, and then a global Coefficient of discharge independent on the type of flow, which can be used for general cases of natural Leaky Woody Dams. For this analysis, the flow discharges and depths over the logs is separated from the pressure flows obtained at the crest in each log (i.e., the increment of flow under pressure when overtopping is not taken into account).

An important conclusion from these results is that the C_d of discharge is almost independent of the submergence or Δh between upstream and downstream. For same discharge, for example 10 l/s, the depths upstream are similar in both free flow or submerged flow conditions for smooth or natural logs Smooth-U = 103 mm,

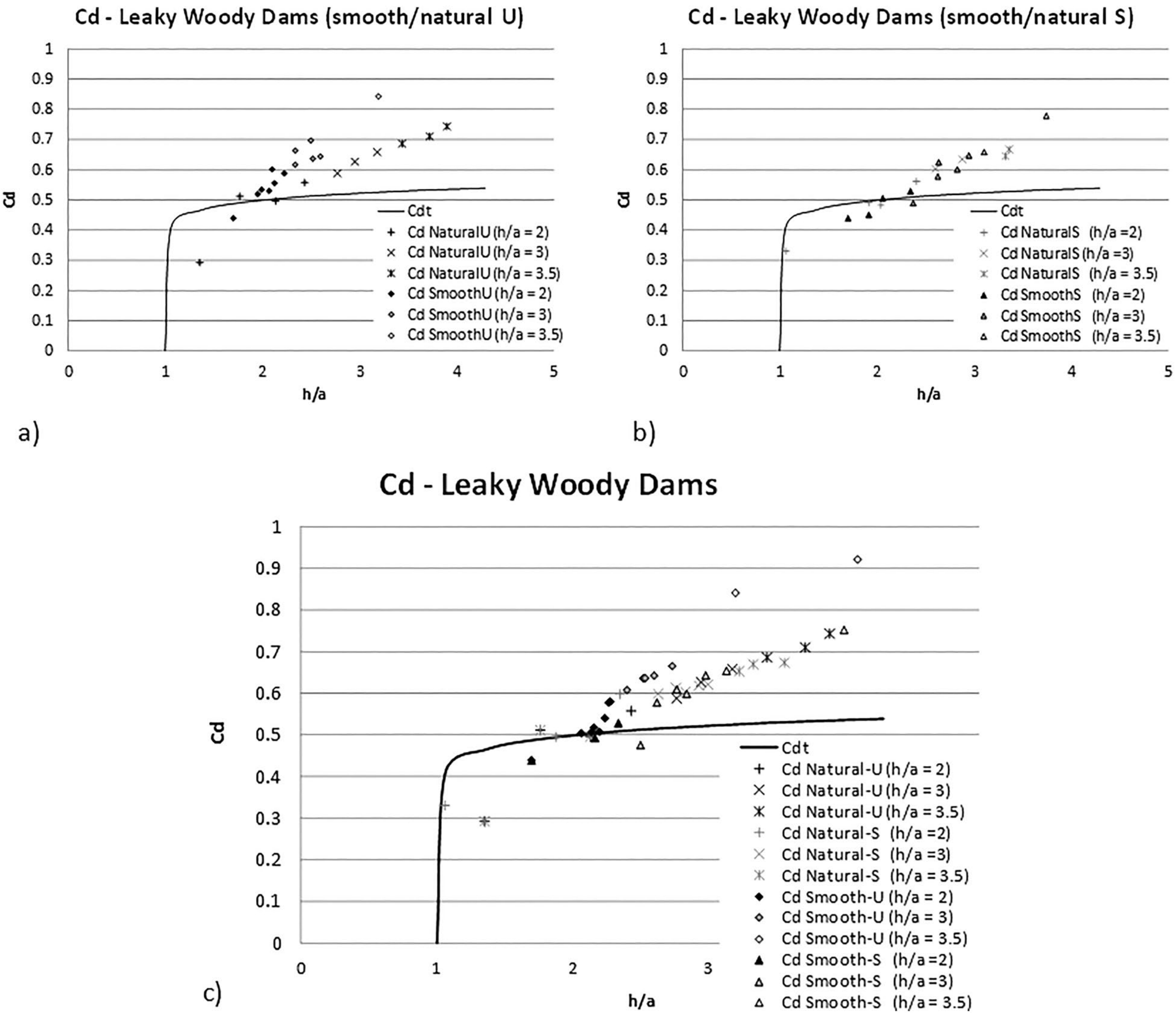

FIGURE 11 | (a) C_d for smooth and natural logs in Unsubmerged conditions. (b) C_d for smooth and natural logs in Submerged conditions. (c) C_d for all cases.

TABLE 4 | Summary of results for smooth/natural logs with unsubmerged/submerged flow conditions. D_{press} is depth under pressure. C_{dt} is the theoretical coefficient of discharge.

Type	h (mm)	h_0 (mm)	Δh (mm)	Q (l/s)	D_{press}	h/a	C_{dt} theory	C_d exp
Smooth- U	17–155	13–57	4–98	1–20	10/30/50	1.7–4.1	0.48–0.54	0.44–0.92
Smooth- S	17–149	13–118	4–31	1–16	"	1.7–4.0	0.48–0.54	0.44–0.75
Natural- U	23–152	17–45	6–107	1–15	17/31/41	1.4–3.9	0.46–0.53	0.29–0.74
Natural- S	18–151	18–111	3–40	1–15	"	1.0–3.6	0.46–0.53	0.33–0.67

Smooth-S = 107 mm, Natural-U = 111 mm and Natural-S = 109 mm. The only variable that affects C_d is the depth upstream the logs and the opening of gaps under pressure. The reason for that is because in both conditions, Unsubmerged or Submerged, there is significant flow downstream of the logs, so the total head is the gravitational head plus the velocity head. This total head only depends on the head losses, or the Cd of the logs, which depends only on the discharge flowing through the logs. This means that in Bernoulli equation (Equation 2) the value $h^2 + V^2/2g$ is almost the same in both Unsubmerged and Submerged conditions.

5 | Coefficients of Discharge for Leaky Woody Dams. Discussion

The experimental results show that the equation for flow under a sluice gate is not fully applicable to flow through leaky dams, as the pressure C_d clearly increases with depth, while the empirical C_d in the literature suggests a trend to a constant value for relative depths (h/a) bigger than 2. Also, the effect of overtopping for each individual log is not taken into account with this equation. For overtopping flows, the different logs are isolated, so the

C_o (coefficient of overtopping) calculated is for each individual log. Using Equation (2), the values of C_o for the discharges overtopping the logs as free flow can be calculated and graphically represented together with the C_d for pressure flows. Figures 12 and 13 show the values of C_o for Smooth and Natural logs in both Unsubmerged and Submerged conditions.

Figure 12 shows that the values of C_o are smaller than the values of pressure C_d and they reach an almost constant value of C_o = 0.5–0.6 for higher overtopping depths. This different behavior of C_o in part counteracts the high values of C_d when the pressure flow is maximum at the crest of each log. An averaged value for a Total C_d (Equation 3) is estimated for most of the

TABLE 5 | Tests for natural logs with unsubmerged flow conditions downstream (Natural-U). C_d is the coefficient of discharge for pressure flows. D_{over} is the depth overtopping.

Q (l/s) natural–U	*h* (mm)	h_0 (mm)	Δh (mm)	Type of flow	D_{over} (mm)	D_{press} a(mm)	*h*/*a*	C_{dt} theory	C_d *exp*
1	23	17	6	Log1 pressure flow		17	1.35	0.4635	0.2920
2	30	24	6	Log1 pressure flow		17	1.76	0.4892	0.5114
3.3	42	29	13	Log1 pressure flow		17	2.47	0.5112	0.7131
4	57	21	36	Log 1 overtopping	16	17	3.35	0.527	0.7420
4	58	21	37	Log 1 overtopping	17	17	3.41	0.5278	
4.3	60	22	38	Log 1 overtopping	19	17	3.53	0.5293	
5	64	25	39	Log 2–1 pressure		31	2.13	0.5024	0.4960
6	73	28	45	Log 2–1 pressure		31	2.43	0.5104	0.5573
6.8	81	28	53	Log 2–1 pressure		31	2.70	0.5161	0.5996
7	88	26	62	Log 2 overtopping	8	31	2.93	0.5204	
7.6	93	27	66	Log 2 overtopping	13	31	3.10	0.5232	
8	95	29	66	Log 2 overtopping		41	3.17	0.5242	
9	102	30	72	Log 2 overtopping		41	3.40	0.5276	
10	108	33	75	Log 2 overtopping		41	2.77	0.5174	0.5874
11	115	36	79	Log 3–1 pressure		41	2.95	0.5207	0.6262
12	124	38	86	Log 3–1 pressure		41	3.18	0.5244	0.6578
13	134	40	94	Log 3–1 pressure	11	41	3.44	0.5281	0.6856
14	145	42	103	Log 3 overtopping	18	41	3.72	0.5317	
15	152	45	107	Log 3 overtopping	22	41	3.90	0.5338	

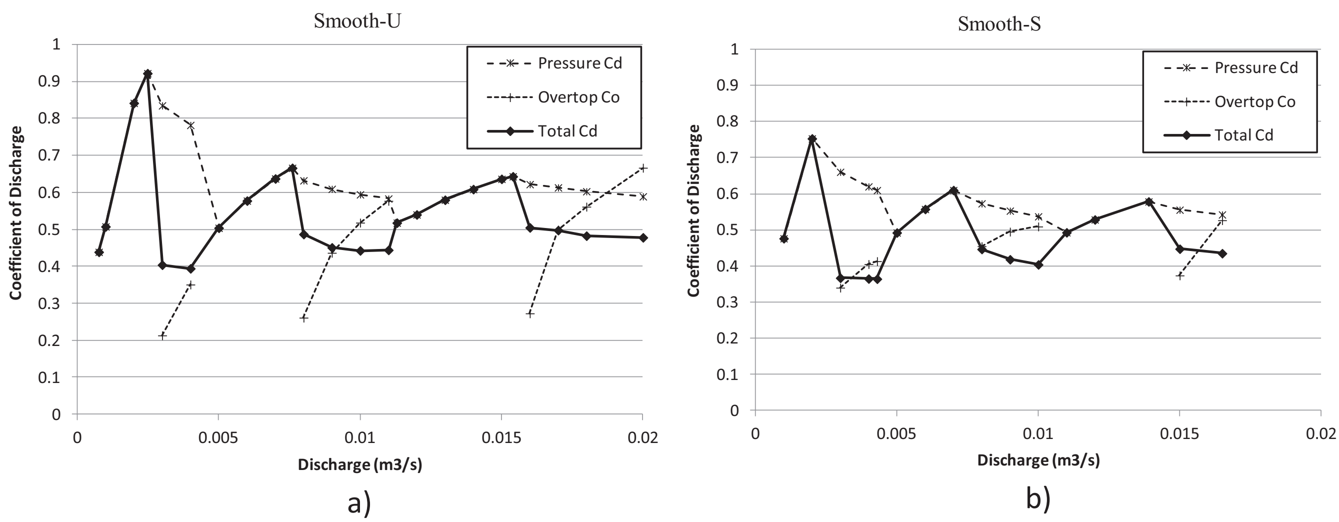

FIGURE 12 | Values of C_d for pressure flow C_o for overtopping flow and Total C_d for (a) Smooth-U logs and (b) Smooth-S logs.

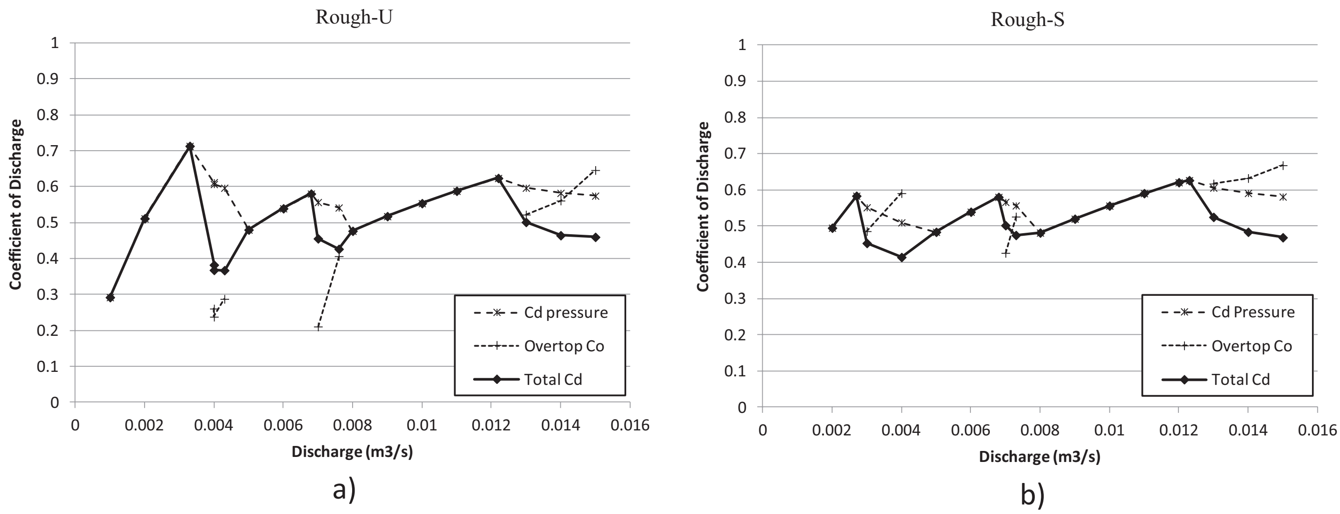

FIGURE 13 | Values of C_d for pressure flow C_o for overtopping flow and Total C_d for (a) Natural-U logs, and (b) Natural-S logs.

discharges as $C_d = 0.5–0.6$ in both Unsubmerged and Submerged cases for smooth logs.

Figure 13 shows the same C_o values and averaged C_d (Total C_d) for natural cases in both Unsubmerged and Submerged conditions. The same pattern as for smooth cases is found, and the value for C_d is also between 0.5–0.6 in most of the cases, except for low pressurized discharges under logs. These cases are less important as the leaky dam does not yet work for diverting water to floodplain, storing water, or slowing the flow.

The analysis of two different C_d, one for pressure flow conditions and another one for overflow conditions do not show any practical advantage, as it adds complexity to the analysis of flow in LWD for practical applications. However the use of a Total C_d with a range of values that can be used easily in modeling it is very practical for real applications in LWD modeling and analysis. The equation proposed in this research is a combination of pressure flow under a small orifice and flow overtopping in a weir:

$$Q = b \cdot a_i \cdot C_d \cdot \sqrt{2g} \cdot h^{1/2} \quad (3)$$

$$\text{with } C_d = 0.5 - 0.6$$

where h = head energy upstream of the LWD (usually is equal to depth upstream of the LWD), a_i = sum of depth of openings and overtopping depth of LWD, b = width of the orifice and overtopping flow areas, and C_d = total coefficient of discharge for the LWD.

The main limitation of these results is that they cannot overly generalize as they are based on limited experimental conditions, in a small-scale flume with a straight geometry and smooth roughness. The field conditions of LWD are different from the experimental conditions due to the wide variety of geometries in natural LWD. These results require validation with field work in the future.

6 | Conclusions

This research provides theoretical and empirical evidence of the hydraulic behavior of leaky woody dams (LWD), commonly used as nature-based solutions for flood risk management, adding more knowledge of their hydraulic performance and modeling. From the literature review and hydraulic analysis, it can be concluded that no practical methods are generally agreed upon for the assessment of LWD and that new datasets are needed for future catchment-scale assessments of LWD. In this research, flume experiments were conducted to observe flow variations and to determine the key parameters affecting the flow and backwater rise in LWD. The experiments were performed in the laboratory to allow for a systematic analysis of discharge coefficients under controlled conditions. Both smooth and natural logs were used to compare natural characteristics in logs with experimental models.

The flow dynamics through natural LWD were analyzed, and a model is proposed for practical applications to better represent the behavior of natural LWD. The proposed model in this paper has been tested using experimental measurements. A simple flow equation under a sluice gate is suggested, with a total coefficient of discharge (C_d) applicable to both flow under pressure and overtopping flow. Four different scenarios were tested: smooth logs in submerged and unsubmerged conditions downstream and natural logs in submerged and unsubmerged conditions downstream. All four scenarios were modeled using the same equation, and a general C_d value between 0.5 and 0.6 is proposed. These results should be validated with field measurements and 3D modeling to confirm their applicability in 2D modeling of LWD.

More experimental research using different log materials and configurations (isolating the effect of each parameter) is necessary to better understand the hydraulic behavior between water flow and leaky dams. The production of new experimental datasets will be important to calibrate and validate numerical simulation of LWD in real applications.

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Conflicts of Interest

The authors declare no conflicts of interest.

Data Availability Statement

Data available on request from the authors.

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