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Local Scour in Bridge Piers: A Reliability-Based Approach

Rajarshi Pramanik[1] · **G. L. Sivakumar Babu**[2] · **Alok Bhowmick**[3]

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Abstract As a vital structure in civil engineering, the performance assessment of bridges becomes very imperative in view of several bridge failures around the globe. Scouring is one of the major causes of the failure of bridges that needs to be taken care of during design. Therefore, the present study aims to investigate scouring in bridge piers under the probabilistic framework. The required pier diameters in achieving three levels of target reliability indices ($\beta_T = 3.0$, 4.0, and 5.0) are estimated for different foundation depths and angles of attack of the flow considered in the study. The load and resistance factors corresponding to the same target reliability indices are also back-calculated from the design points obtained from the First Order Reliability Method. The influence of variabilities of the input parameters on the computed pier diameters and load and resistance factors is also examined. The parametric study shows that the foundation depth is the most influencing parameter in determining the required pier diameter, followed by the coefficient for streambed conditions and flow velocity. From the analysis, it is found that for $\beta_T = 3.0$, the pier diameter is increased by around 41.5% for an increase in the mean foundation depth of 1.0 m (from 4 to 5 m). Further, the study suggests that the effect of flow angles on the estimated pier diameter is also significant. For example, keeping the target reliability index and coefficient of variation (COV) of input variables at a constant value, the required pier diameter is reduced by around 60%, as the angle of attack of the flow is increased by four times. Finally, one design problem is presented to ensure the efficiency of the present approach, and recommendations for the current practice followed in India are provided.

Keywords Reliability analysis · Scour depth · Pier diameter · LRFD · Target reliability index · Coefficient of variation

Introduction

Bridges are one of the relevant structures in civil engineering infrastructures that provide the connection between routes crossing over rivers or canyons. Being an important structure, it is warranted that engineers from all civil engineering communities need to take precautions to maintain the safety of bridges. One of the causes of bridge failure occurs due to the scouring of soil particles beneath the piers or abutments. Scouring is an event where erosion of soil particles increases the depth of the riverbed by creating a hole around the bridge foundations. As a result, different hazardous phenomena like settlements, rotations, and sliding of the foundation occur. So, the safety of the bridge deck is hampered by induced additional stresses and deformations, leading to the collapse of the structure [1, 2]. Kamojjala et al. [3] reported that in 1993, out of 28 bridge failures due to flooding of the upper Mississippi, 22 failures were caused due to scouring. From the investigations done by several researchers [4–6], it is concluded that scouring is the main

✉ Rajarshi Pramanik
rajarshi.juconst@gmail.com

G. L. Sivakumar Babu
gls@iisc.ac.in

Alok Bhowmick
bsec.ab@gmail.com

1 Department of Civil Engineering, National Institute of Technology Silchar, Cachar, Assam 788010, India

2 Department of Civil Engineering, Indian Institute of Science, Bengaluru, Karnataka 560012, India

3 B&S Engineering Consultants Pvt. Ltd., Noida, Uttar Pradesh 201301, India

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governing incident that causes bridge failures. Therefore, proper maintenance and design are required to minimize bridge failures and maintain the longevity of the bridges. A proper investigation towards defining the adequate depth of scour is one of the most challenging tasks for design engineers at the time of bridge construction. Figure 1 illustrates a schematic diagram of scouring at the bridge pier, defining different parameters associated with the scour event. Total scour depth generally consists of three parts: general, local, and contraction. There is always some scouring phenomenon occurring in the riverbed regardless of the existence of a bridge, which is called a general scour. It degrades the riverbed continuously, resulting in lowering of the riverbed level. Local scour is generated after the construction of bridge piers and abutments due to the vortex formed by the flow velocity. Finally, the formation of the contraction scour happens due to the narrowing of the riverbed width caused by the presence of abutments and piers. This contraction increases the flow velocity and its erosive capacity. In this paper, local scour is considered for the analysis, as it is the most vulnerable condition experienced during bridge failures.

To date, several scour models [7–16] any others have been developed to estimate the scour depth around the bridge piers or abutments. Kothyari et al. [8] proposed an equation to compute the maximum scour depth around the circular bridge piers during live-bed condition. The study also investigated the effect of unsteadiness of the flow on the estimated scour depth. Sheppard and Miller Jr. [11] performed the experimental investigations on a circular pile for a range of water depths and flow velocities. The local scour depths measured from the experimental

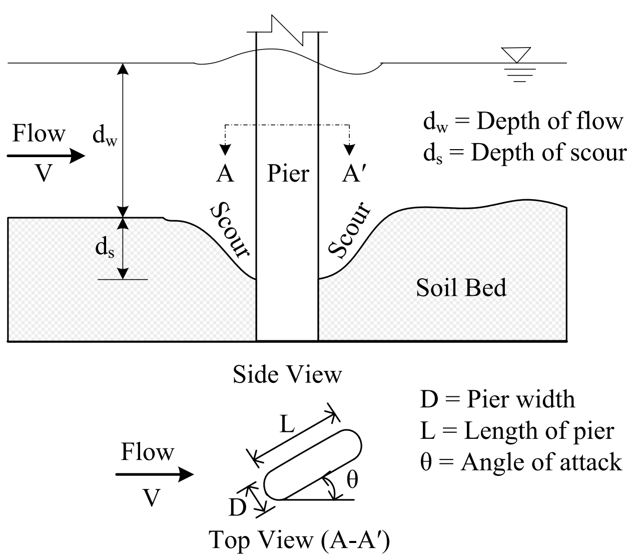


Fig. 1 Schematic diagram of scouring at bridge pier of round nose shape

investigations were compared with the depths estimated through the previously developed equations. Further, Debnath and Chaudhuri [12] carried out a laboratory experiment to estimate the local scour at circular bridge piers embedded in a clay-sand-mixed bed. The effects of clay content, water content, and particle size on the estimated maximum scour depth, and geometry of the scour hole were observed for the sand in the prepared clay-sand mixture. A regression-based equation was also developed as functions of pier Froude number, clay content, water content, and bed shear strength. Other research works are also involved in predicting the estimation of the scour depth under different river bed conditions and flow conditions. However, these models are deterministic in nature and unable to capture the uncertainties associated with the river and bridge characteristics. Nevertheless, it is worth mentioning that the safety of any structure can only be fully ensured when it comes under the probabilistic framework. Till now, very few studies [2, 17–22] have been performed regarding the bridge scour under the probabilistic platform. Johnson et al. [19] quantified both parameter and model uncertainties using the scour equations. The overall reliability of the scour equations was also assessed in the study. Kallias and Imam [20] performed the statistical analysis of the expected maximum annual flow of rivers and simultaneously carried out the Monte Carlo simulation to estimate the probability of local scour failure. Results were presented through a case study and the effect of climate changes had been incorporated through the estimation of the maximum annual flow. Shahriar et al. [22] developed a framework for reliability-based pier scour assessment methodology. The Load and Resistance Factor Design (LRFD) approach was also applied to determine the scour factors and the developed approach was demonstrated through three illustrative examples of axially and laterally loaded pile design. In all studies regarding probabilistic bridge scour analysis, the performance of the existing bridges (mostly case studies) is assessed. To the best of the author's knowledge, no general design guidelines under the probabilistic framework have been proposed till now. So, the reliability-based design (RBD) of scour in bridge piers is scanty and needs to be addressed in detail.

The First Order Reliability Method (FORM) method involves an optimization technique to search for the most probable point (MPP). Unlike the Second Order Reliability Method (SORM), this method is much simpler and involves less computational time without compromising the accuracy of the results. Hence, the present study deals with a reliability-based design (RBD) of scour in bridge piers using FORM. In this regard, several design charts are presented in terms of the required pier width (diameter for the round nose shape) corresponding to the three levels of target reliability index of 3.0, 4.0, and 5.0. The load and resistance factors are also back-calculated from the design points obtained from FORM. The effects of variabilities of different input parameters on the estimated pier diameter and load and resistance factors are analyzed. A number of design charts are also presented for different angles of attack of the flow experienced by the piers. Finally, the effectiveness of the present investigation is shown using a design problem, and recommendations for the current practice available in India using IRC: 78–2014 [23] are provided.

Deterministic Model for Estimation of Scour Depth

Among different scour models cited in the previous section, HEC-18 design equation [14] is adopted in this paper for the prediction of the local scour in bridge piers due to its wide acceptability by various researchers [2, 17, 18, 20]. Also, this model is developed considering all probable parameters resulting in the development of scour near the vicinity of the bridge piers. According to the model, the scour depth is estimated as below [20].

$$d_s = 2.0 d_w K_1 K_2 K_3 K_4 \left(\frac{D}{d_w}\right)^{0.65} F_0^{0.43} \tag{1}$$

where d_s is the maximum scour depth in bridge piers, d_w is the flow depth. K_1, K_2, K_3 and K_4 are the coefficients indicating pier shape, angle of attack, streambed conditions, and the river bed material size, respectively. K_1 depends on the pier nose shape. In the present study, it is taken as 1.0 for the round nose. For other shapes, one can refer to Arneson et al. [14]. K_2 considers the effect of the angle of attack of the river flow computed by [14],

$$K_2 = \left(\cos\theta + \frac{L}{D}\sin\theta\right)^{0.65} \tag{2}$$

where θ is the angle of attack of the flow, and L is the length of the pier taken as 18.0 m. If L/D is larger than 12.0, then L/D is taken 12.0 as a maximum value. K_2 is equal to 1.0 for zero angle of attack.

K_3 depends on the bed condition. For the plane bed and clear water scour, it is taken as 1.1. Different values of K_3 corresponding to different bed conditions can be found in Arneson et al. [14]. K_4 depends on the diameter of the river bed material. It varies from 0.4 (for fine material) to 1.0 (for coarse material). For the present study, it is taken as 1.0. D is the pier width (for the round nose pier section, it is same as the pier diameter), and F_0 is the Froude number determined as follows:

$$F_0 = \frac{V}{\sqrt{g d_w}} \tag{3}$$

where g is the acceleration due to gravity taken as 9.81 m/s^2, and V is the velocity of flow given by:

$$V = \frac{Q}{B d_w} \tag{4}$$

where Q is the discharge of flow, B is the river width. Depth of flow, d_w can be obtained by [20]:

$$d_w = \left(\frac{nQ}{B\sqrt{s}}\right)^{\frac{3}{5}} \tag{5}$$

where n is the Manning's coefficient and s is the longitudinal slope of the channel.

In terms of the velocity of flow, Eq. 5 can be modified by:

$$d_w = \left(\frac{nV}{\sqrt{s}}\right)^{\frac{3}{2}} \tag{6}$$

Performance Function and Reliability Index

First, the performance function needs to be defined before conducting the reliability analysis. For bridge scour reliability analysis, the performance function is defined by the difference between the foundation depth and estimated scour depth from the deterministic model, which is given by,

$$g(\mathbf{X}) = d_f - d_s = d_f - \left[2.0 d_w K_1 K_2 K_3 K_4 \left(\frac{D}{d_w}\right)^{0.65} F_0^{0.43}\right] \tag{7}$$

where d_f is the foundation depth. $\mathbf{X}$ is a vector of system design variables consisting of uncertain parameters. Failure will occur for $g(\mathbf{X}) \le 0$ states, and the reliability index (β) is determined using the First Order Reliability Method (FORM) as discussed below.

Calculation of the reliability index is a minimization problem. As proposed by Hasofer and Lind [24], β is defined as the minimum distance from the origin to the failure surface in the standard normal space. In general, β is formulated as follows [25].

$$\beta = \min_{\mathbf{X}\in F} \sqrt{\left(x_i - \mu_i^N\right)^T \left[\mathbf{C}^N\right]^{-1} \left(x_i - \mu_i^N\right)} \tag{8}$$

where F is the failure domain, $\mathbf{C}^N$ is the covariance matrix for the equivalent normal standard deviation $\boldsymbol{\sigma}^N$. μ_i^N and σ_i^N are the equivalent normal mean and standard deviation of the variable. For nonnormal variables, equivalent mean and standard deviation are computed as follows.

$$\sigma^N = \frac{\phi\{\Phi^{-1}[F(x)]\}}{f(x)} \tag{9}$$

$$\mu^N = x - \sigma^N \Phi^{-1}[F(x)] \tag{10}$$

where x = original nonnormal variate, $\Phi^{-1}[\bullet]$ = inverse of the cumulative distribution function (CDF) of a standard normal distribution, $F(x)$ is the original nonnormal CDF evaluated at x, $\phi\{\bullet\}$ is the probability density function (PDF) of the standard normal distribution, and $f(x)$ is the original non-normal probability density ordinate at x.

Alternatively, Eq. 8 can be rewritten as

$$\beta = \min_{\mathbf{X}\in F} \sqrt{\left(\frac{x_i - \mu_i^N}{\sigma_i^N}\right)^T [\mathbf{R}]^{-1} \left(\frac{x_i - \mu_i^N}{\sigma_i^N}\right)} \tag{11}$$

where **R** is the matrix of correlation coefficients between the standard normal variables. For a graphical representation of the reliability index, one can refer to Low and Tang [25, 26], and Sivakumar Babu and Singh [27].

LRFD Approach

In recent times, LRFD approach has gained popularity among researchers [27–33] over the allowable stress design (ASD) approach, where the factor of safety concept has been used to incorporate all uncertainties regarding the variation of applied loads and the resistance of the structure to bear the loads.

In this section, the LRFD concept is briefly summarized. The basic principle of the LRFD approach is shown in Fig. 2 [27]. The basic relationship between factored load and resistance is explained by [32],

$$\sum (\mathrm{LF})_i Q_{ni} \le (\mathrm{RF}) R_n \quad \text{or} \quad \sum \gamma_i Q_{ni} \le \varphi R_n \tag{12}$$

where LF = γ_i = load factors; RF = φ = resistance factor; Q_{ni} = nominal loads; R_n = nominal resistance.

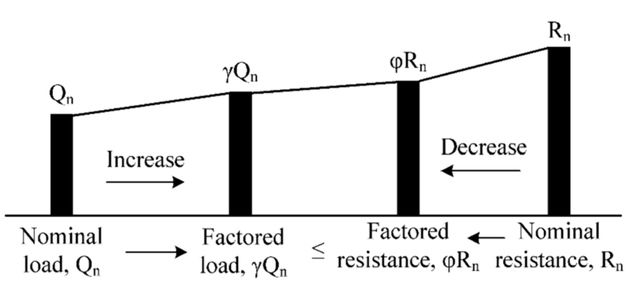


Fig. 2 Illustration of LRFD approach

To achieve the most economical design, Eq. 12 takes the form of

$$\sum (\mathrm{LF})_i Q_{ni} = (\mathrm{RF}) R_n \quad \text{or} \quad \sum \gamma_i Q_{ni} = \varphi R_n \tag{13}$$

alternatively,

$$\sum \text{Design (factored) values of load} = \text{Design (factored) value of resistance} \tag{14}$$

RBD Versus. LRFD

In this section, the merits and demerits of the LRFD and reliability-based design (RBD), and also comparability between these two approaches, are discussed to give a clear idea about the complementary role of RBD toward LRFD to the design practitioners. It is noteworthy that the present study only concerns how RBD can complement LRFD, not to replace it.

a) In RBD, the design point is determined when the design resistance is equal to the design loads on the limit state surface (LSS). The design point obtained from the RBD is similar to the LRFD design point as shown in Eq. 14. Nevertheless, it is worth mentioning that the LRFD design point cannot predict the parameter uncertainties and sensitivities of the parameters, but in the RBD, it can be done.
b) In RBD, the target reliability index and the corresponding probability of failure can be estimated, whereas it is unknown for the LRFD. In LRFD, for most of the cases, the conservative design is achieved by defining the nominal load and resistance values.
c) Reliability-based design requires the statistical values (mean, standard deviation, etc.) of random variables to achieve a target reliability index, whereas, for the LRFD, only conservative nominal load and resistance values need to be defined for the analysis. However, it is unknown how much conservativeness is required for a safe design. Therefore, being a flexible and robust technique, the RBD acts as a complementary role to a partial factors design approach (Eurocode 7 [34]).

The detailed discussion on this section is not presented here for brevity, which can be found in Low [32].

Reliability-Based Design for Scour Depth

In this section, one example problem of scouring in bridge piers is presented through RBD via FORM using MATLAB [35]. In this RBD approach, the required pier diameter (D) is estimated to achieve the target reliability indices of 3.0, 4.0, and 5.0. In addition, the load and resistance factors are back-calculated from the design points obtained from FORM. As mentioned by Low [32], the estimated load and resistance factors are case-specific and nonintrinsic.

In this example problem, most of the parameters regarding river material and bridge pier geometry are taken from Kallias and Imam [20]. The mean velocity of flow is only assumed to be 2.30 m/s. Table 1 shows the complete list of both deterministic and probabilistic parameters considered in this study. COV values of the random variables are mostly taken from Kallias and Imam [20], but some COV values are assumed to be higher than that reported by Kallias and Imam [20] to account for the higher variability. However, effect of the variation in COVs of the parameter is highlighted later in this paper. Herein, the depth of foundation is also taken as a random variable with varying mean and standard deviation.

Estimation of Pier Diameter, Load and Resistance Factors

In this sub-section, the reliability results are presented for varying mean values of the foundation depth (μ_{df}) for three levels of target reliability index of 3.0, 4.0, and 5.0. Initially, an exercise is performed for the angles of attack of the flow (θ) equal to 0º and 20º; the effect of θ on the reliability results is presented later in this paper. This paper aims to estimate the dimension of the pier diameter (D) to achieve β_T of 3.0, 4.0, and 5.0, and subsequently, the back-calculated load and resistance factors (from the design points of the FORM analysis) are also presented.

In LRFD, the back-calculated load and resistance factors are highly dependent on the nominal values of load and resistance terms. In this paper, load and resistance terms are scour depth (d_s) and foundation depth (d_f), respectively. It can be noted that the nominal load and resistance are higher and lower than the mean values of the load and resistance terms, respectively. In this study, the nominal values of load and resistance are taken to be at $\mu \pm 1\sigma$.

Table 2 presents the estimated pier diameter corresponding to the different mean values of the foundation depth to achieve a sufficiently safe design in terms of a target reliability index β_T of 3.0, 4.0, and 5.0. Two values of the angle of attack of the flow ($\theta = 0$º and 20º) are selected for each case. The pier diameter value is adjusted to a suitable value for attaining β_T of 3.0, 4.0, and 5.0. For $\mu_{\text{df}} = 4.0$ m, and $\theta = 0$º, it can be noticed that the estimated pier diameter values are 1.35, 1.09, 0.88 m for $\beta_T = 3.0$, 4.0, and 5.0, respectively (see Table 2). As discussed above, the nominal foundation depth is kept as $\mu_{\text{df}} - \sigma_{\text{df}}$; for example, the nominal foundation depth is equal to $4.0 - 0.4 = 3.60$ m for $\mu_{\text{df}} = 4.0$ m (for $\text{COV}_{\text{df}} = 10\%$). Furthermore, the mean value of the scour depth is calculated from Eq. 1 for the values presented in Table 1, and 10^6 number of realizations using the Monte Carlo simulation (MCS) is run to obtain the standard deviation of the scour depth. The selection of the optimum number of realizations has been done using the procedure followed by Pramanik et al. [36]. The results are not presented here for brevity. Therefore, the nominal value of the scour depth is computed as $\mu_{\text{ds}} + \sigma_{\text{ds}}$; for example, it is calculated as $2.515 + 0.773 = 3.288$ m for $\mu_{\text{df}} = 4.0$ m, $\theta = 0$º, and $\beta_T = 3.0$. The design depth of the foundation (d_f^*) is obtained from the FORM analysis, which is equal to the design scour depth (d_s^*) (see Table 2). Hence, the load factor (LF_{ds}) is computed as $\gamma_{\text{ds}} = d_s^*/d_{\text{sn}} = 3.221/3.288 = 0.98$ for $\mu_{\text{df}} = 4.0$ m, $\theta = 0$º, and $\beta_T = 3.0$. Similarly, the resistance factor (RF_{df}) is calculated as $\varphi_{\text{df}} = d_f^*/d_{\text{fn}} = 3.221/3.60 = 0.$ 895 for the same values of μ_{df}, θ, and β_T. Likewise, other

Table 1 Statistical properties of variables considered for the study

Variables		Mean	COV (%)	Type	Probability distribution
River	Width (m)	65.0	10	Probabilistic	Lognormal
	Mean velocity of flow (m/s)	2.30	10	Probabilistic	Lognormal
	Coeff. for streambed conditions (K_3)	1.1	5	Probabilistic	Uniform
	Coeff. for bed material size (K_4)	1.0	–	Deterministic	–
	Slope	0.0032	5	Probabilistic	Lognormal
	Manning's coefficient	0.035	28	Probabilistic	Lognormal
Bridge piers	Foundation depth (m)	4.0	10	Probabilistic	Lognormal
	Coeff. for pier nose shape (K_1)	1.0	–	Deterministic	–
	Coeff. for angle of attack (K_2)	1.0[a]	–	Deterministic	–

[a]the mean value is shown for angle of attack of the flow, $\theta = 0$º. For other θ values, it will be calculated as $K_2 = (\cos\theta + L/D \sin\theta)^{0.65}$ where L = length of pier

Table 2 Reliability-based design of pier diameter D for target reliability index β_T of 3.0, 4.0, and 5.0 and back calculated load and resistance factors

μ_{df} (m)	θ (º)	β_T	D (m)	Nominal d_f (m)	μ_{ds} (m)	σ_{ds} (m)	Nominal d_s (m)	$d_f^* = d_s^*$ (m)	LF_{ds} (γ_{ds})	RF_{df} (φ_{df})
4.0	0	3.0	1.35	3.60	2.515	0.773	3.288	3.221	0.980	0.895
		4.0	1.09	3.60	2.188	0.673	2.861	2.983	1.043	0.829
		5.0	0.88	3.60	1.904	0.585	2.489	2.758	1.108	0.766
	20	3.0	0.27	3.60	2.517	0.774	3.291	3.223	0.979	0.895
		4.0	0.22	3.60	2.188	0.672	2.860	2.982	1.043	0.828
		5.0	0.18	3.60	1.908	0.586	2.494	2.761	1.107	0.767
4.5	0	3.0	1.62	4.05	2.831	0.870	3.701	3.625	0.979	0.895
		4.0	1.31	4.05	2.466	0.758	3.224	3.359	1.042	0.829
		5.0	1.06	4.05	2.149	0.661	2.810	3.108	1.106	0.768
	20	3.0	0.32	4.05	2.830	0.870	3.700	3.625	0.980	0.895
		4.0	0.26	4.05	2.462	0.757	3.219	3.356	1.043	0.829
		5.0	0.21	4.05	2.148	0.660	2.808	3.108	1.107	0.767
5.0	0	3.0	1.91	4.50	3.151	0.969	4.120	4.032	0.979	0.896
		4.0	1.54	4.50	2.740	0.842	3.582	3.732	1.042	0.829
		5.0	1.25	4.50	2.392	0.735	3.127	3.457	1.105	0.768
	20	3.0	0.38	4.50	3.147	0.968	4.115	4.029	0.979	0.895
		4.0	0.31	4.50	2.738	0.842	3.580	3.730	1.042	0.829
		5.0	0.25	4.50	2.387	0.734	3.121	3.453	1.107	0.767

entries in Table 2 are estimated. It can be observed that the change in the required pier diameter for achieving β_T of 3.0, 4.0, and 5.0 is significant. As β_T increases, the required pier diameter decreases because the possibility of scouring is reduced with decreasing pier diameter, which will help to achieve a higher safety margin. Moreover, the pier diameter is also decreased by a subsequent amount as the angle of attack of the flow increases from 0 to 20º. However, the pier diameter increases with an increase in the foundation depth from 4.0 to 4.5 m or 4.5 to 5.0 m. For example, for $\beta_T = 3.0$, and $\theta = 0$º, the pier diameter is increased by around 20% and 18% from an increase in the mean foundation depth from 4.0 to 4.5 m and 4.5 to 5.0 m, respectively. So, a total of around 41.5% increment in the pier diameter value is observed for an increase in the mean foundation depth of 1.0 m. On the other hand, changes in the load and resistance factors are mainly significant for increasing β_T values; however, these changes are trivial for other cases, i.e., changes in mean foundation depth and angle of attack of the flow. This observation implies that the pier diameters against different β_T values are self-adjusted despite changing mean foundation depth and angle of attack of the flow. Moreover, LRFD does not provide any information regarding the probability of failure (P_f) of the system. For example, for $\mu_{df} = 4.0$ m, $\theta = 0$º, if the estimated pier diameter is less than 1.35 m using the LRFD approach (with its nominal values, LF and RF and satisfying Eq. 13), the implied reliability index will be higher than 3.0 ($P_f < 0.135\%$). Similarly, if the estimated pier diameter is greater than 1.35 m, the implied reliability index will be smaller than 3.0 ($P_f > 0.135\%$). However, in this study, the load and resistance factors are back-calculated from the FORM design points.

Effect of Variation in Input Parameters

This section describes the effect of the COV values of different random variables considered in this study on the estimated pier diameter and the back-calculated load and resistance factors. In this regard, COVs of all input parameters are varied from 5 to 20% except Manning's coefficient (n), whose COV is varied from 5 to 28%, as the maximum value is already taken for the previous analysis. Figure 3 plots the variation in the required pier diameter with different COVs of the parameters against β_T values of 3.0, 4.0, and 5.0. It can be seen that the required pier diameter decreases with an increase in the COVs of the input parameters for the same level of β_T. It indicates that as COVs of the input parameters increase, the variability level also increases and chances of scouring in bridge piers are increased. Therefore, the pier diameter decreases to maintain the specified target reliability indices as presented in the paper. Also, for a particular COV of the input variables, the pier diameter is decreased by a notable amount with an increase in the β_T value. The same explanation is also valid for this observation. However, the effect of variation in the COVs of slope and Manning's coefficient on the estimated pier diameter is negligible, as no significant difference in the pier diameter values is observed for varying COVs of those two parameters. It can also be

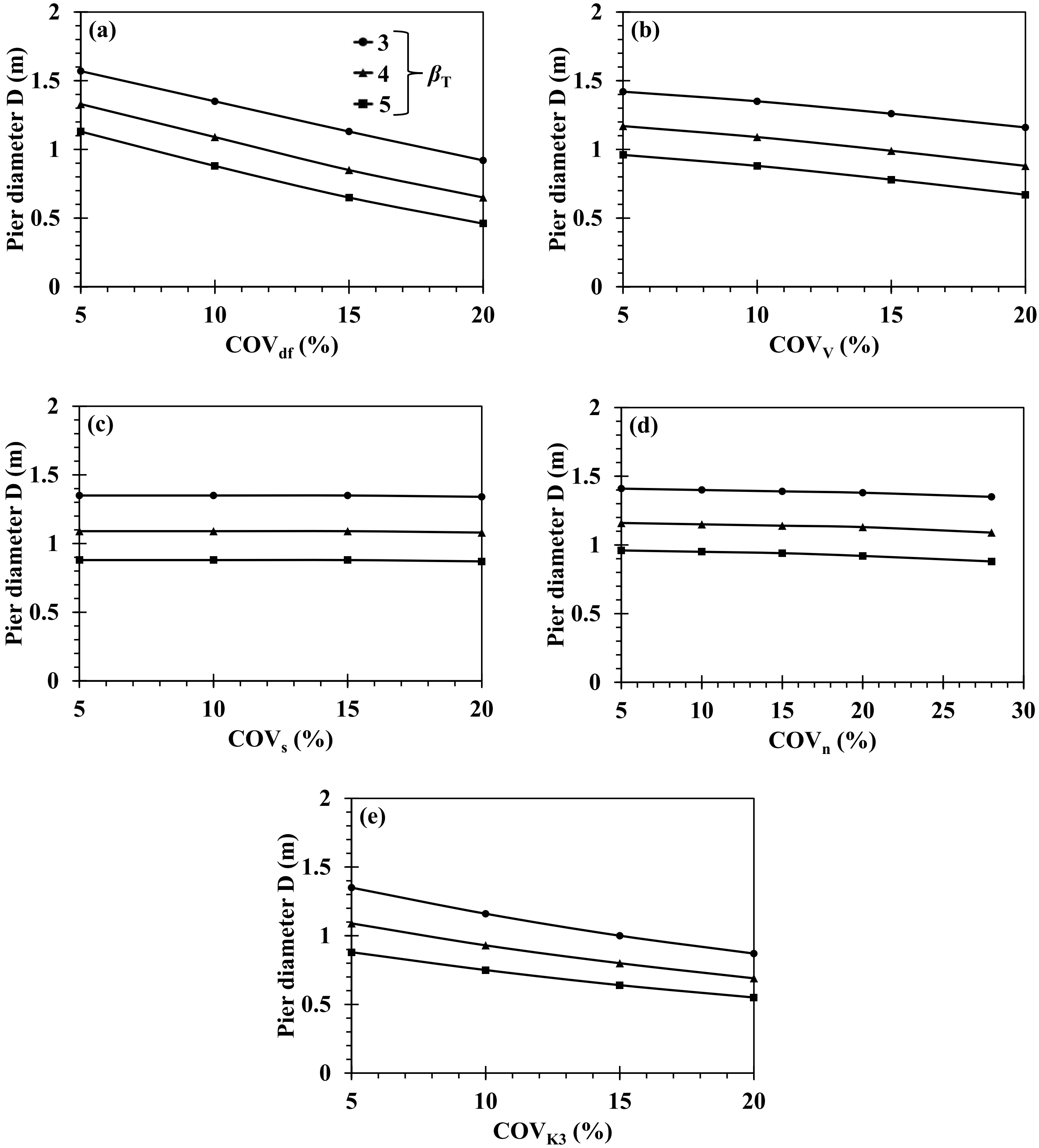


Fig. 3 Variation of pier diameter (D) against different target reliability indices for changing COVs of (**a**) foundation depth d_f; (**b**) mean velocity of flow V; (**c**) slope of the channel s; (**d**) Manning's coefficient n; (e) coefficient K_3

noticed that the effect of variation in the COVs of the depth of foundation (d_f) and coefficient K_3 is significantly higher than other parameters. For example, to achieve $\beta_T = 5.0$, the pier diameter is reduced by 59.3% and 37.5%, as COV's of d_f and coefficient K_3 are increased from 5 to 20%, respectively. In contrast, the pier diameter is reduced by 30.2%, as COV's of V is increased from 5 to 20% for the same β_T value. It is worth mentioning that the parametric study presented above is an indirect technique to indicate the sensitivity of the input parameters on the output.

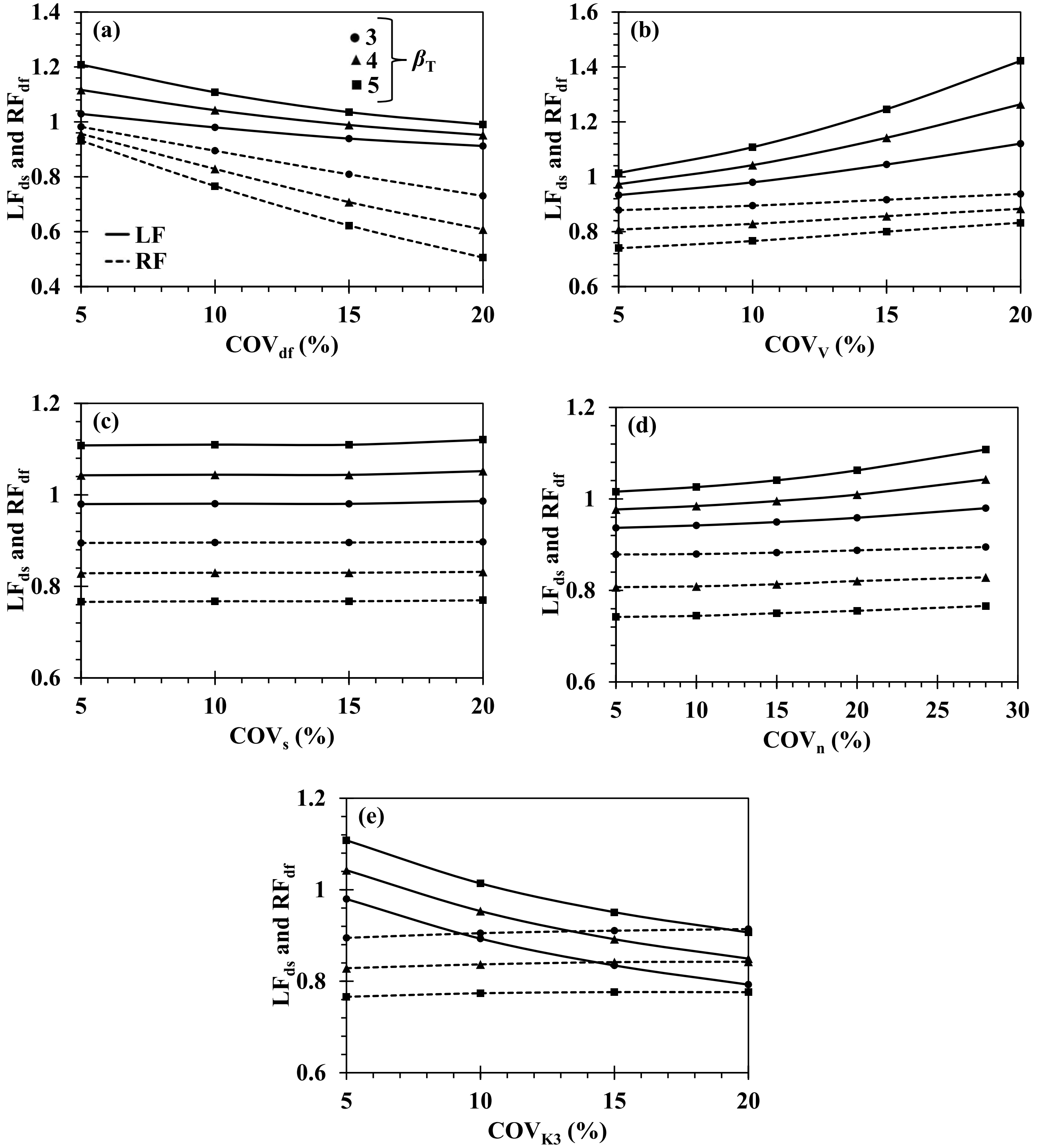


Fig. 4 Variation of load and resistance factors against different target reliability indices for changing COVs of (a) foundation depth d_f; (**b**) mean velocity of flow V; (**c**) slope of the channel s; (**d**) Manning's coefficient n; (**e**) coefficient K_3

Further, the influence of COVs of the input parameters on the load and resistance factors is studied and plotted in Fig. 4. It can be observed that the load factors increase with an increase in the target reliability index. In contrast, the reverse phenomenon is observed for the resistance factors. The first row in Table 2 shows that the load and resistance factors are 0.98 and 0.895, respectively for $\beta_T = 3.0$. For a higher target reliability index ($\beta_T = 4.0$ or 5.0), the required pier diameter is smaller (1.09 and 0.88 m, respectively), the back-calculated load factor is larger (1.043 and 1.108, respectively), and the back-calculated resistance factor is smaller (0.829 and 0.766, respectively), as summarized in Table 2 for $\mu_{df} = 4.0$ m and $\theta = 0^\circ$. This observation is similar to the results obtained from the strip footing case reported by Low [32]. For $COV_{df} = 10\%$, the ratios of the load and resistance factors are equal to 1.10, 1.26, and 1.45 for $\beta_T = 3.0$, 4.0, and 5.0, respectively (see Fig. 4a). It can be noted that the load and resistance factors are back-calculated from the design points of FORM analysis and are highly dependent on the selected nominal load and resistance values. So, the obtained load and resistance factors and also ratios between them are nonintrinsic and case-specific. The results will differ for different nominal values of load and resistance. Further, it can be observed that if 80 percentile is used for the nominal load and 20 percentile for the nominal resistance, then the back-calculated load factor will be larger and resistance factor will be smaller than the present case, where 84 percentile has been used for the nominal load and 16 percentile has been used for the nominal resistance. Alternatively, it can be concluded that if less conservative nominal values are selected, then the back-calculated load and resistance factors will be more conservative, and vice versa. However, RBD is free from this assumption, as no nominal values are required as inputs. Here also, the effect of slope and Manning's coefficient on the calculated LF and RF is insignificant, as the changes in load and resistance factor values are negligible.

Effect of Angle of Attack of the Flow

In most of situation, it is experienced that the bridge pier sections make an angle to the direction of the flow, which affects the stability of the piers. Therefore, a study to figure out the effect of the flow angle on the pier's stability is essential for the safe design of the bridge piers on the river. Hence, in this section, the effect of the angle of attack of the flow on the required pier diameter achieving target reliability index of 3.0, 4.0, and 5.0 is presented for different COVs of the random parameters (Figs. 5, 6, 7, 8 and 9). This exercise provides a number of design charts in terms of the pier diameter against different flow angles and β_T values. It can be noted that the COV values vary from 5 to 20% for all the variables other than Manning's coefficient n. COV of n varies from 5 to 28%, similar to the previous results (Figs. 3d, 4d). When COV of one variable is changing, COVs of other variables are kept at a constant value, as shown in Table 1. It can further be noted that Fig. 5 presents the variation of the pier diameter with different flow angles for all four COV values of foundation depth, i.e., 5, 10, 15, and 20%. Other figures (Figs. 6, 7, 8, and 9) present the same except for the COVs shown in Table 1, which is already presented in Fig. 5b. From the figures, it can be observed that the required pier diameter reduces with increasing flow angles and target reliability index values. It also decreases with increasing COVs of the input parameters. The same explanation as discussed in the previous section is also valid here. As the angle of attack increases, the bridge piers are exposed to a vulnerable condition and chances of scouring in bridge piers are increased. Therefore, the required pier diameter is reduced to maintain the specified target reliability indices as presented in the paper. A similar phenomenon also occurred when the COVs. Moreover, the depth of foundation (d_f) has the maximum effect on determining the required

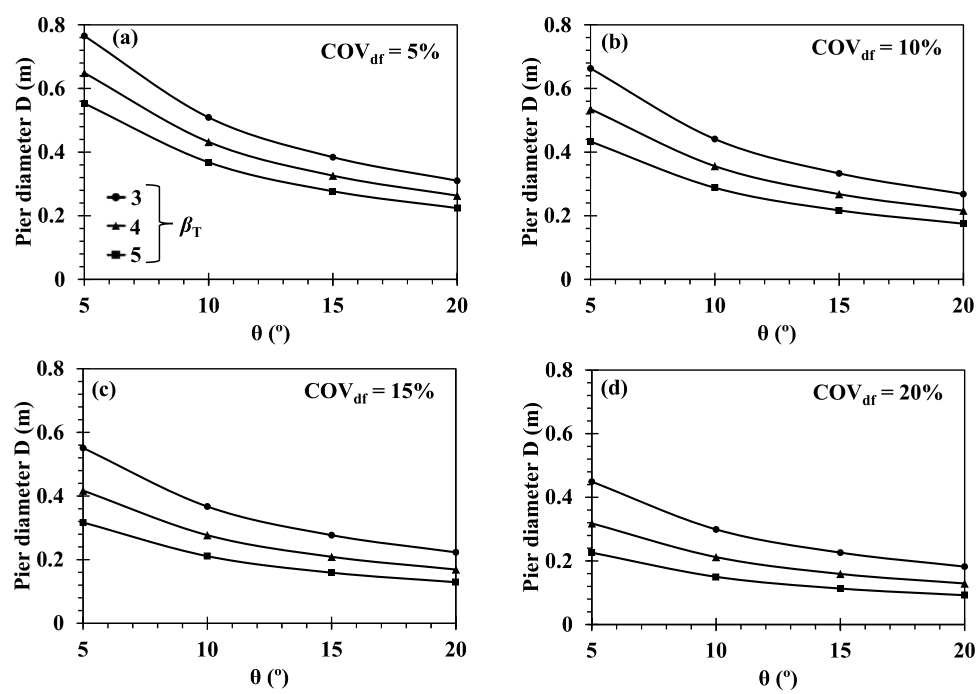


Fig. 5 Variation of pier diameter (D) with different values of angle of attack of the flow (θ) for COV of foundation depth (**a**) 5%, (**b**) 10%, (**c**) 15%, (**d**) 20%

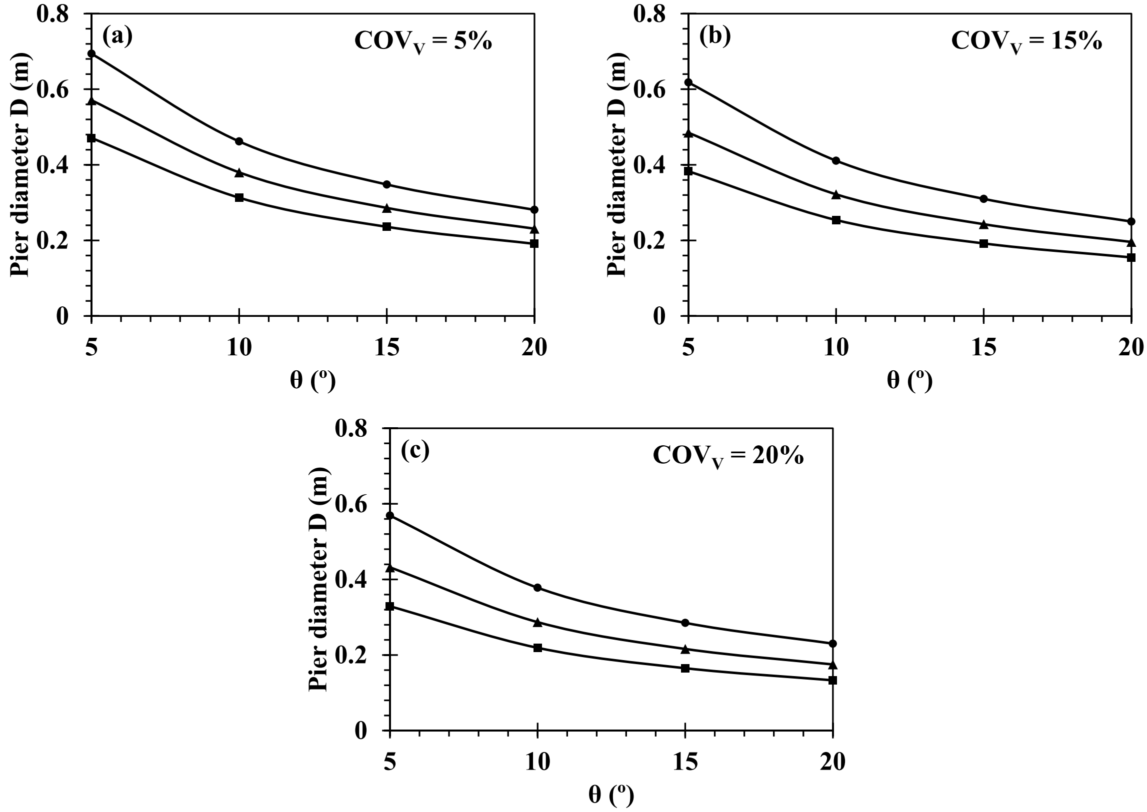


Fig. 6 Variation of pier diameter (D) with different values of angle of attack of the flow (θ) for COV of mean velocity of flow (**a**) 5%, (**b**) 15%, (**c**) 20%

pier diameter for different β_T values. It is followed by coefficient K_3, and mean velocity of the flow (V). For example, for $\beta_T = 3.0$, and $COV_{df} = 20\%$, the pier diameter is reduced by around 60%, as the angle of attack of the flow is increased by four times (from 5 to 20º). Furthermore, for $\beta_T = 3.0$, and $\theta = 10º$, the pier diameter is reduced by nearly 32%, as COV_{df} is increased twice (from 10 to 20%). In addition, for $\theta = 10º$, the pier diameter is reduced by almost 41% and 48% for $\beta_T = 4.0$, and 5.0, respectively, for the same increment in the COV_{df} value (see Fig. 5). Moreover, for $\theta = 10º$, the pier diameter is reduced by nearly 36.1%, 37.1%, and 37.5% for $\beta_T = 3.0$, 4.0, and 5.0, respectively, as COV_{K3} is increased by four times (from 5 to 20%) (see Figs. 5b, and 9). It can be concluded that the rate of reduction in the pier diameter with increasing β_T values is less for coefficient K_3 compared to the foundation depth. Again, for $\theta = 10º$, the pier diameter is reduced by near about 18.2%, 24.5%, and 30% for $\beta_T = 3.0$, 4.0, and 5.0, respectively, as COV_V is increased by four times (from 5 to 20%) (see Fig. 6). From this part of the study, it can be concluded that as flow angle increases the chances of scouring also increase, therefore the required pier diameter is reduced by a substantial amount to maintain the target reliability index to a specified value as mentioned in this paper.

Design Example

Let us study a design problem where the pier diameter needs to be determined for the bridge piers where the foundation depth is kept as 4.5 m. All other parameters are kept the same as shown in Table 1 for flow angle (θ) equal to 0º. Taking the factor of safety of 1.5, the maximum scour depth should not exceed 3.0 m. So, the required pier diameter for this condition is calculated as 1.77 m (from Eq. 1). From the current reliability-based study, the pier diameter is estimated as 1.62 m for β_T equal to 3.0. The corresponding load and resistance factors are 0.979 and 0.895, respectively. It can be seen that the estimated pier diameter is less than the computed pier diameter from the formulation provided by the HEC-18 [14]. Therefore, it can be concluded that the RBD always gives an economical design outcome. For higher target reliability indices, one can refer to Table 2 for the required pier diameter. Moreover, as presented in this study, design practitioners can follow different design chats to account for the uncertainties in input parameters and different angles of attack of the flow.

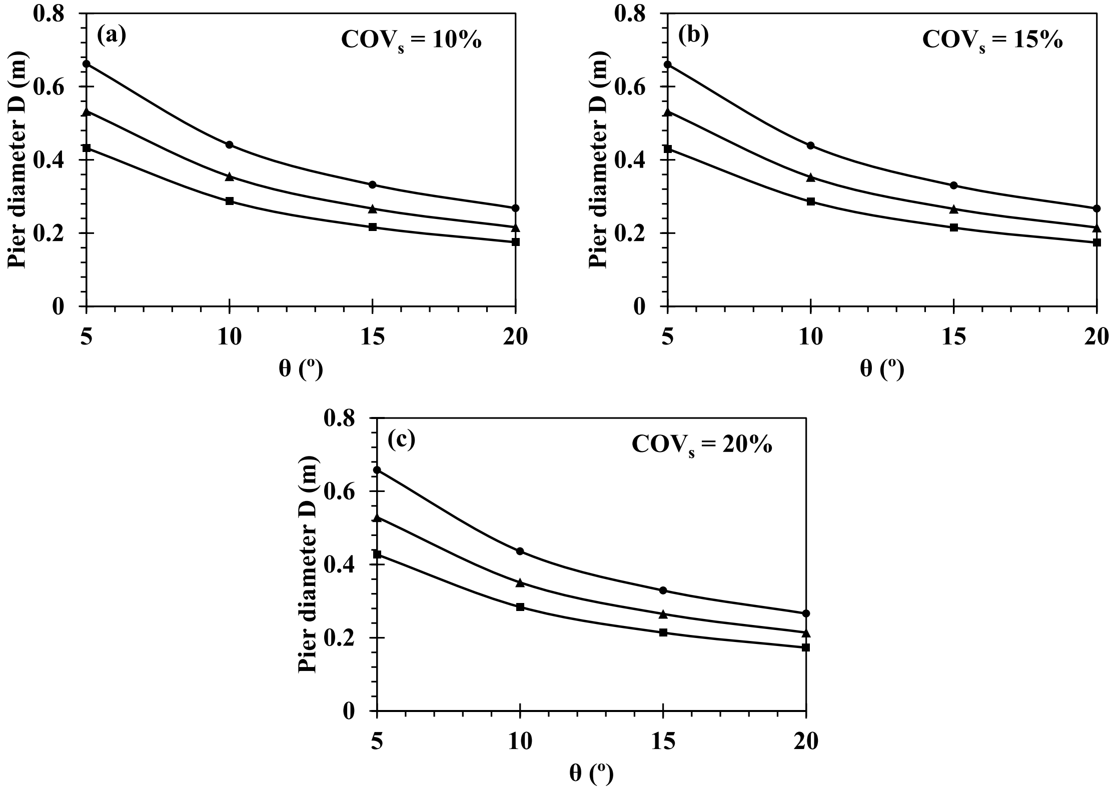
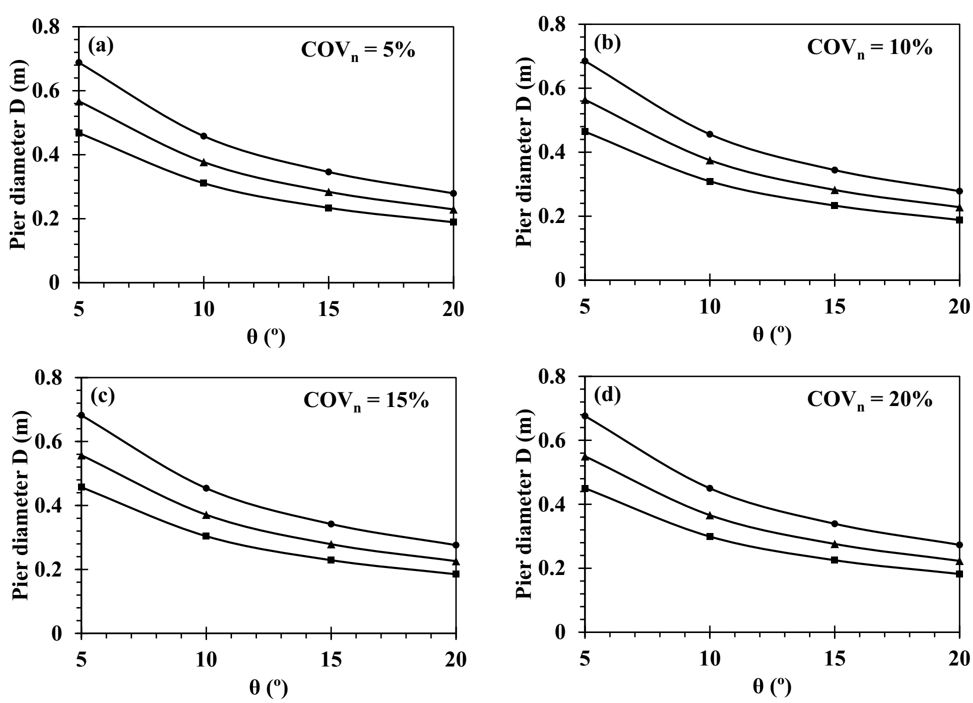


Fig. 7 Variation of pier diameter (D) with different values of angle of attack of the flow (θ) for COV of slope (**a**) 10%, (**b**) 15%, (**c**) 20%

Fig. 8 Variation of pier diameter (D) with different values of angle of attack of the flow (θ) for COV of Manning's coefficient (**a**) 5%, (**b**) 10%, (**c**) 15%, (**d**) 20%



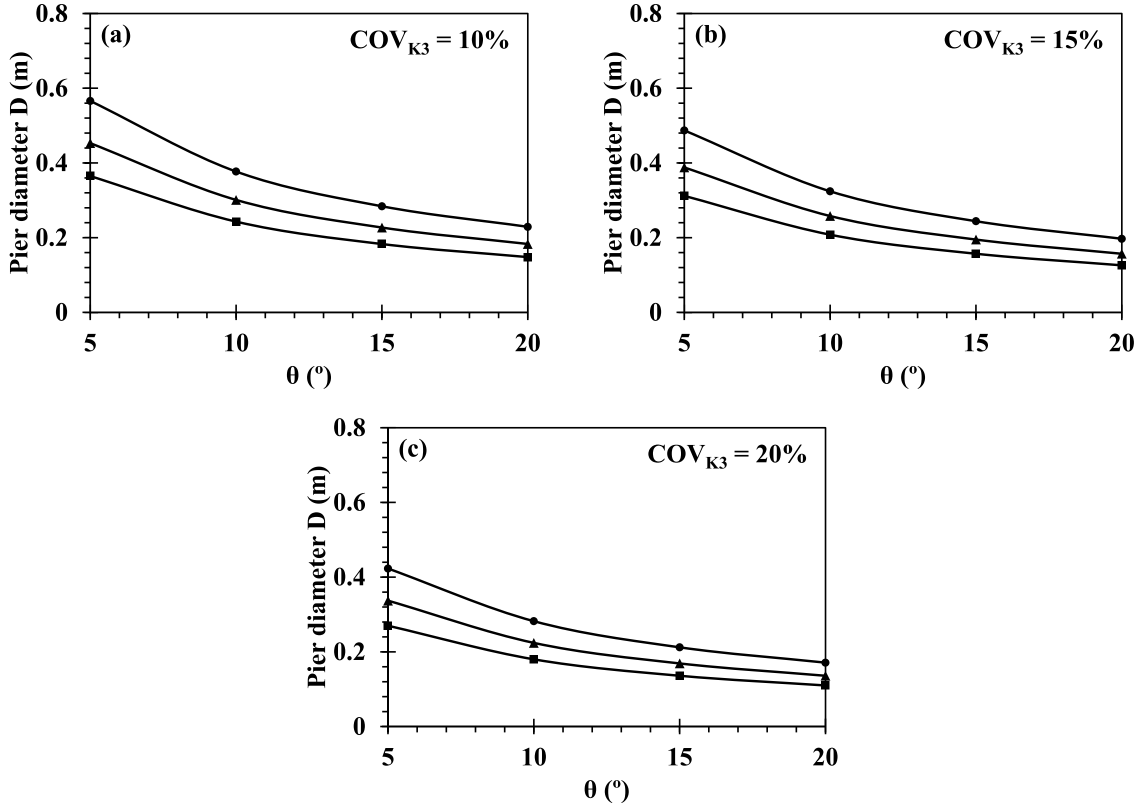


Fig. 9 Variation of pier diameter (D) with different values of angle of attack of the flow (θ) for COV of coefficient K_3 (**a**) 10%, (**b**) 15%, (**c**) 20%

Comments on IRC Method

Indian Road Congress (IRC) uses the Lacey's [37] equation for estimating the scour depth in a pier. According to IRC: 78–2014 [23], the scour depth is determined as below.

$$d_s = 1.34\left(\frac{q^2}{k_{sf}}\right)^{\frac{1}{3}} \tag{15}$$

where d_s = normal scour depth in m; q = discharge in m^3/s per m width; k_{sf} = silt factor for material obtained up to the deepest anticipated scour, which is equal to $1.76\times(d_m)^{0.5}$, d_m is the weighted mean diameter of particles in mm. For different types of soils, values of q and k_{sf} can be found in IRC: 78–2014 [23]. The total scour depth at piers can be estimated as $2d_s$.

It can be seen that many factors like depth of flow, pier diameter, and different coefficients, as incorporated in Eq. 1 are not considered in the above equation for the estimation of the bridge scour. Mainly, the IRC method does not provide any information regarding the pier geometry or modification coefficient for the pier shape, which is indeed a significant drawback of the IRC method. Therefore, IRC recommendations may give erroneous results compared to the HEC-18 formulation. For example, for all the values shown in Table 1 and for pier diameter 1.0 m, HEC-18 formulation (Eq. 1) gives the scour depth equal to 2.07 m. Whereas, from the IRC method, it is estimated as $2\times2.15 = 4.30$ m (taking $k_{sf} = 1.5$), which is nearly 2.1 times higher than the HEC-18 method. Hence, it can be concluded that the IRC method needs to be revised incorporating all the factors into its scour depth calculation formulation.

Conclusions

The present study demonstrates a reliability-based design for scouring in bridge piers. For the sake of safe design, the required pier diameters for the specified levels of target reliability indices are estimated through the RBD. Further, the load and resistance factors are back-calculated for the different foundation depths and flow angles from the design points obtained from the FORM analysis. The influence of input parameters on the estimated pier diameter is illustrated throughout the study. From the parametric study, it has been observed that to achieve $\beta_T = 5.0$, the pier diameter is reduced by 59.3%, 37.5%, and 30.2% as COV's of the depth of foundation, coefficient K_3, and velocity of the flow are increased from 5 to 20%, respectively. Hence, it can be concluded that the depth of foundation is the most influencing parameter in determining the pier diameter against the target reliability indices, followed by the coefficient K_3 and velocity of the flow. The study also presents the variation of the pier diameter against different angles of attack of the flow experienced by the bridge piers. From the analysis, it is observed that for a particular target reliability index and COV's of the input parameters, the pier diameter is reduced by around 60%, as the angle of attack of the flow is increased by four times (from 5 to 20º). Therefore, it can be concluded that the flow angle significantly affects the computed pier diameter, which warrants proper engineering analysis before designing the bridge piers. From the results, it can also be concluded that the rate of reduction in the pier diameter with increasing angle of attack is highest for changing variability in the foundation depth. For a particular target reliability index and angle of attack, the reduction in the pier diameter is nearly about 48% as the COV of the foundation depth is increased from 10 to 20%. The reduction is less for changing variability of other parameters. The effect of the depth of foundation is more significant here also. The study also points out the shortcomings that exist in the IRC Method as many parameters are not considered while calculating the scour depth using the IRC Method yields a higher value of the scour depth than that predicted using the most popular HEC-18 formulation. Finally, the use of the design charts presented in the study is demonstrated through a design problem, and the purpose for a reliability-based design is well justified.

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Declarations

Conflict of interest The authors have no relevant financial or non-financial interests to disclose.

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