CHAPTER 217

A REVIEW OF WAVE/CURRENT-INDUCED SCOUR AROUND PIPELINES

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ABSTRACT

A comprehensive review is presented of scour around pipelines in the case of non-cohesive sediment. The review is organized in four main sections, namely the two-dimensional scour, the three-dimensional scour, the effect of scour on forces on and vibrations of pipelines and the mathematical modelling of scour process. Over sixty works were included in the review.

1. INTRODUCTION

Marine pipelines are used for disposal of industrial and municipal wastewater into the sea, for cooling water in nuclear power plants, and for the transportation of gas and crude oil from offshore platforms. Marine cables, on the other hand, are increasingly used for communication.

Design of marine pipelines and marine cables with regard to their stability is a rather complicated problem. One of the factors which needs to be taken into consideration in the design process is the scour around the pipeline.

The scour around the pipeline is caused by the very presence of the pipeline itself. There are numerous aspects of the problem which need to be addressed during the design process, such as the formation of spans (particularly in determining the maximum extent of spans as well as timing of any remedial action), the self-burial of pipelines, the effect of self-burial on the pipeline stability, just to give a few examples.

A large volume of knowledge has been accumulated on the subject during the last decade or so, as a result of intensive research activities in countries such as Holland, Norway, U.K., U.S.A., Denmark and several others. The purpose of the present paper is to review this research work. Only the non-cohesive sediment bed is considered.

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2. TWO-DIMENSIONAL SCOUR

Onset of scour

Consider a pipeline which is laid on an erodible bed. Assume that the bed, soil and flow conditions are the same along the length of the pipe, i.e. the conditions are strictly two-dimensional. If the initial embedment of the pipeline is not very large and if the flow around the pipe is sufficiently strong, scour may break out underneath the pipe. The stage at which the scour breaks out is called the onset of scour.

The onset of scour is directly related to the seepage flow which occurs in the sand beneath the pipe, caused by the pressure difference between the upstream and the downstream of the pipe (Fig. 1c). When the flow velocity is gradually increased, a critical point is approached where the discharge of the seepage flow is increased more rapidly than the driving pressure difference dictates. Simultaneously the surface of the sand at the immediate downstream of the pipe rises and eventually a mixture of sand and water breaks through the space underneath the pipe. This phenomenon is called piping and is well known in soil mechanics in hydraulic structures such as dams, cofferdams etc. (Terzaghi, 1948).

The conditions under which the onset of scour occurs below pipelines have been studied by Mao (1986) and Chiew (1990) in steady currents. Mao described the role of separation vortices that form in front and at the rear of the pipe in the process of the onset of scour. Also, he discussed the seepage flow underneath the pipe in relation to the onset of scour. The latter has been further elaborated by Chiew. Chiew further linked the onset of scour to the phenomenon of piping.

In the case of waves, the piping conditions are created underneath the pipe in the same way as in steady currents. The action is immediate. However, if the conditions are such that the critical condition is not attained immediately, then the action of separation vortices will become important to create scour more early in the wave case. Sumer & Fredsøe (1991) has linked the onset of scour in waves to the latter through the Keulegan-Carpenter number and expressed the critical condition for the onset of scour by the following empirical equation

$$\frac{e_{cr}}{D} = 0.1 \ln(KC) \tag{1}$$

in which e_{cr} = the critical embedment of the pipe beyond which no scour occurs, D = the pipe diameter and KC = the Keulegan-Carpenter number defined by

$$KC = \frac{U_m T_w}{D} \tag{2}$$

in which U_m = the maximum value of the orbital velocity of water particles at the bed, T_w = the wave period. For a sinusoidal motion of water particles KC will obviously be

$$KC = \frac{2\pi a}{D} \tag{3}$$

in which a = the amplitude of the orbital motion of water particles at the bed.

Time development of scour

The onset of scour is followed by the stage of so-called *tunnel erosion* (Leeuwenstein et al., 1985; Hansen et al., 1986). Sumer et al. (1990) present data regarding bed shear stress just under the pipe corresponding to this stage. The data indicate that the bed shear stress is

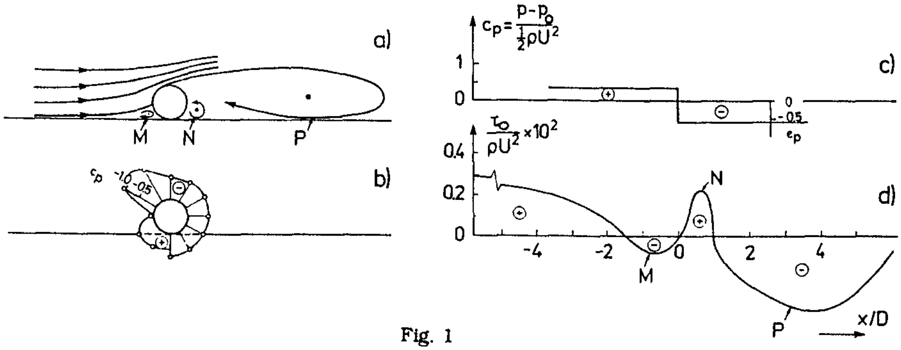
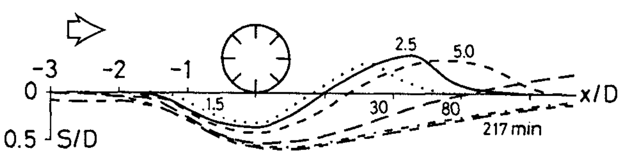


Fig. 1



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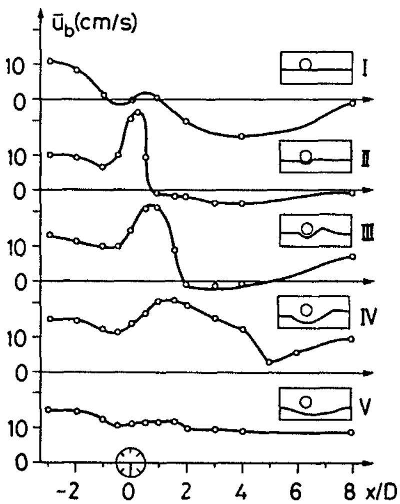

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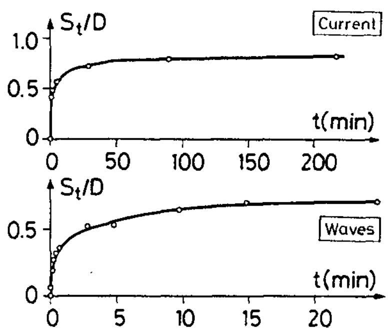

Fig. 4

increased by a factor of 4 with respect to its undisturbed value, resulting in an increase in the sediment transport rate of a factor of 8. This explains why the scour below the pipe occurs so fast and so violently during the tunnel erosion.

The tunnel erosion is followed by the stage called *lee-wake erosion*. In this stage, the scour downstream of the pipe is governed by the lee-wake of the pipe. Sumer et al. (1988b) made an investigation of the effect of lee-wake on scour. This work demonstrates that 1) vortex shedding is present in the lee-wake from rather early stages of the scouring process, and 2) the scour downstream of the pipe is eventually governed by this organized flow.

Fig. 2 shows a typical example where the time development of scour can be seen. The mild slope of the downstream portion of the scour profile is due to the action of the pipe's lee-wake.

As seen from the figure, the scour process attains an equilibrium stage where scouring below the pipe comes to an end. When this stage is reached, the sediment transport will be the same at all sections over the reach of the scour hole, and therefore the amount of sediment which enters the scour hole will obviously be identical to that leaving the scour hole.

Fig. 3 shows the results of Jensen et al.'s (1990) velocity measurements very close to the bed at different stages of the scour process. The figure shows that, while the velocity below the pipe is increased tremendously at the initial stage of the scour process (Profile II), it eventually becomes practically identical to the undisturbed flow velocity, as the scour approaches towards its equilibrium stage (Profile V).

The depth of the scour hole is probably the most significant quantity. Normally, it is taken as the depth of the scour hole just below the pipe.

The scour depth develops towards the fully-developed equilibrium stage through a transitional period, as illustrated in Fig. 4 for a pipe rigidly placed on a bed with initially zero gap. The depth corresponding to the fully-developed stage is called the *equilibrium scour depth*. It is also seen from the figure that a certain period of time must be elapsed for a substantial amount of scour to develop. This time is called the *time scale* of scour process.

The equilibrium scour depth, as well as the time scale of the scour process, constitute two major parameters in scour studies. The remaining part of this section will focus on these two parameters.

Equilibrium scour depth

Scour depth has been studied extensively in the case of *steady currents* (Chao & Hennessy (1972), Kjeldsen et al. (1973), Littlejohns (1977), Herbich (1981), Bijker & Leeuwenstein (1984), Lucassen (1984), Leeuwenstein et al. (1985), Herbich (1985), Herbich et al. (1984), Bijker (1986), Ibrahim & Nalluri (1986), Mao (1986), Kristiansen (1988) and Kristiansen & Tørum (1989)).

Kjeldsen et al. (1973) was the first to conduct scour experiments under controlled conditions. The experiments were done in live-bed situations. Kjeldsen et al.'s data indicated that the equilibrium scour depth can be expressed by the following relation:

$$S = 0.972\left(\frac{V^2}{2g}\right)^{0.2} D^{0.8} \qquad (4)$$

in which V = the mean flow velocity. This relation suggests that

$$\frac{S}{D} \propto \theta^{0.2} \qquad (5)$$

in which θ = the Shields parameter defined by

$$\theta = \frac{U_f^2}{g(s-1)d} \tag{6}$$

in which U_f = the bed friction velocity, corresponding to the undisturbed flow, g = the acceleration due to gravity, s = the specific gravity of sediment grains and d = the grain size.

Eq. 5 implies that the normalized scour depth S/D is only a weak function of θ. We shall return to this point later in this section.

The exact flow picture created by the presence of the pipe depends on the following quantities: the pipe diameter D, the flow velocity V, the kinematic viscosity of the fluid ν, the pipe roughness k, and the grain diameter d of the bed material. From these quantities, the dimensionless equilibrium scour depth can be found to depend on the following parameters:

$$\frac{S}{D} = \frac{S}{D}(k^*, R, \theta) \tag{7}$$

in which k^* = k/D is the relative roughness, R = VD/ν is the pipe Reynolds number.

Of the three parameters that appear in Eq. 7, the influence of k^* and the Reynolds number appears through their effect on the downstream flow of the pipe. If the pipe is hydraulically rough, the wake flow is almost unaffected by the Reynolds number, while for a hydraulically smooth pipe some influence of the Reynolds number is expected in the downstream vortex-shedding pattern. Fig. 5 shows a plot of the data by Kjeldsen et al. (1973), Lucassen (1984), Mao (1986) and Kristiansen (1988) on the scour depth below smooth pipes exposed to a current. It is seen in the figure that there is some weak influence of the Reynolds number on the scour depth, because a slight decrease in S occurs for Reynolds number around 10^5 - 3×10^5. For a free circular cylinder, this coincides with the transition from subcritical to supercritical flow (Schewe (1983), Sumer and Fredsøe (1988)). In this transition region, the vortex shedding becomes less pronounced, which might lead to a smaller lee-wake erosion and hence less scour depth.

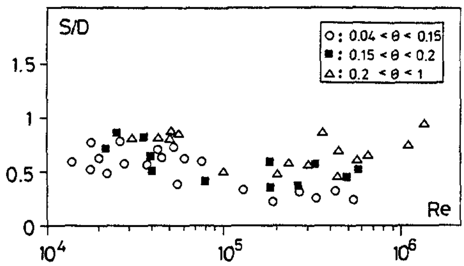

Fig. 5

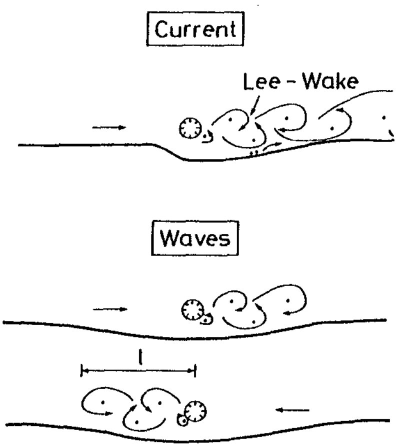

Fig. 6

As far as the influence of θ is concerned, this must be examined in two different categories: the clear-water case, where the sediment far from the pipe is not moving ($\theta < \theta_{cr}$), and the live-bed case, where sediment is transported far from the pipe ($\theta > \theta_{cr}$), in which θ_{cr} = the threshold value of θ for sediment motion. In the clear water case, the variation in scour depth with θ is more pronounced: as S/D increases from 0 at very small θ-values up to values of 0.4 - 1.0 when the θ-value approaches the live-bed case. However, when the live-bed case is obtained, very small variation in S/D is observed, as seen from Fig. 5, as has already been pointed out in conjunction with Kjeldsen et al.'s empirical equation (Eqs. 4 and 5). Ignoring this weak variation it may be suggested that, for all practical purposes, the following relation may be used as the design equation, to predict the equilibrium scour depth in steady currents

$$S/D = 0.6 \pm 0.1 \tag{8}$$

in which the first figure on the right hand side of the equation is the mean of the data plotted in Fig. 5, and the second is the standard deviation.

Regarding the scour in *tidal flows and waves*, the main difference between this case and the steady case is that the downstream-formed wake system now occurs on both sides of the pipeline. Here the strong lee-wake erosion, which gives a much more gentle downstream slope, occurs on both sides of the pipe (Fig. 6). The formation and extension of the wake pattern in oscillatory motion is governed by KC number. Therefore it must be expected that there exists a correlation between the equilibrium scour depth and KC number. Sumer & Fredsøe (1990) demonstrated that this is indeed the case. Based on their experimental data and Lucassen's (1984) data (which were recast in terms of KC number), the following empirical equation was established, relating the equilibrium scour depth to KC number for the live-bed situation ($\theta > \theta_{cr}$) and for a pipe initially in contact with the bed:

$$S/D = 0.1\sqrt{KC} \tag{9}$$

The data suggests that this equation is valid in a very broad range of KC, namely from KC = 2 to about 300. This relation has been confirmed later by Gökçe & Günbak's (1991) experiments and Hansen's (1992) numerical-model results.

Sumer & Fredsøe (1990) discussed also the effect of Shields parameter as well as Re-number dependence plus the effect of the presence of ripples in the wave-flume tests.

Generally, the equations given in the preceding, namely Eqs. 4 or 8 and Eq. 9 may be used as design equations. However, there are several other factors which may influence the scour depth. Of these factors, the following may be mentioned: The pipe position in vertical (Leeuwenstein et al. (1985), Mao (1986), Sumer & Fredsøe (1990)), the roughness of pipe surface (Sumer & Fredsøe (1990)), the angle of attack of the flow (Mao (1988), Hansen (1992)), the current in combined waves and current (Hansen (1992)), the case of multiple pipelines (Westerhorstmann et al. (1992)), the armoring of bed sediment (Sidek & Ibrahim (1992)), vibrations of the pipe (Sumer et al. (1988a)) and the Shields parameter in clear-water scour (Mao (1986), Hansen (1992)).

Time scale

As mentioned previously, the scour depth develops towards its equilibrium stage through a transitional period (Fig. 4), which can be represented by the following relation:

$$S_t = S\left(1 - \exp\left(-\frac{t}{T}\right)\right) \tag{10}$$

The quantity T is defined as the time scale of the scour process and represents the time period during which a substantial scour develops.

Fredsøe et al. (1992) has made a study of this time scale in both steady currents and waves. They found that the time scale normalized as in the following

$$T^* = \frac{[g(s-1)d^3]^{1/2}}{D^2}T \tag{11}$$

is a function of θ, the Shields parameter. Based on their own data as well as the data from Kjeldsen et al. (1973) and Mao (1986), they found that the relation between T^* and θ can be represented by the following simple expression

$$T^* = \frac{1}{50}\theta^{-5/3} \tag{12}$$

for the live bed situation ($\theta > \theta_{cr}$) and for a pipe with an initially zero gap. This equation was found to be valid for both steady currents and waves. Fredsøe et al. (1992) attributed this to the fact that the lee-wake scour -- the key element in the wave induced scour -- is insignificant at the initial stage of the scour process. Therefore the time scale is unable to differentiate whether the flow is a steady-current or a wave-induced flow. In Fredsøe et al. (1992) study, also the effect of change in wave climate has been investigated.

3. THREE-DIMENSIONAL SCOUR

General description

Two-dimensional scour below pipelines, as observed in two-dimensional flume test with a fixed pipe, must in the field turn to three-dimensional scour picture as sketched in Fig. 7 in order to obtain support for the pipeline. Hereby, a three-dimensional scour pattern arises as sketched in Figs. 7a and 7b: a number of scour holes is interrupted by a stretch, where the pipeline is partially or totally buried.

The scour picture in the free-span areas (Section B-B) is two-dimensional, while at other places, particularly in the neighbourhood of span shoulders (Section A-A), it is three-dimensional. See Fig. 8 for a definition sketch.

At Section A-A, the pipe sinks in the soil. This is due to the combined action of three-dimensional scour and soil failure, as will be explained in the following section.

At Section B-B, on the other hand, the scour, after it breaks out, spreads along the length of the pipeline. When the scour hole becomes sufficiently long, the pipe begins to sag into its naturally created trench hole. This may continue until the pipe comes into the neighbourhood of the bottom, which eventually brings an end to the scouring process. From this moment onwards, the backfilling process starts, and later on, the pipeline may partially or fully become covered by sand (self-burial).

Rate of spread of scour along the pipe

The rate of spread of the three-dimensional scour along the pipe is one of the major parameters. Research dealing with this quantity is not extensive. Gravesen & Fredsøe (1983) gave an account of how to deal with the problem when extending the results of model experiments to the nature. Also, various accounts of the spreading process have been given in Leeuwenstein (1985) and Bernetti et al. (1990). Hansen et al. (1991), on the other hand, has presented a semi-empirical model of the process, which can predict the rate of spread of scour along the pipe.

In the case when the current approaches the pipe at an oblique angle, the free span will migrate. This aspect has been investigated by Hansen et al. (1991).

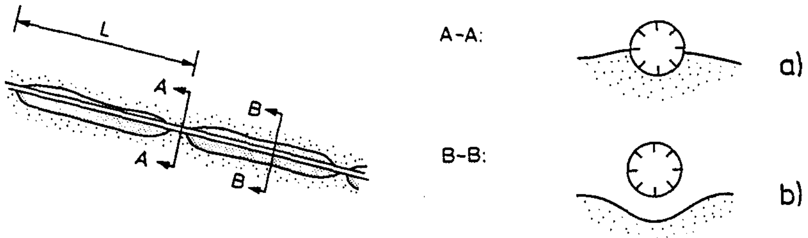

Fig. 7

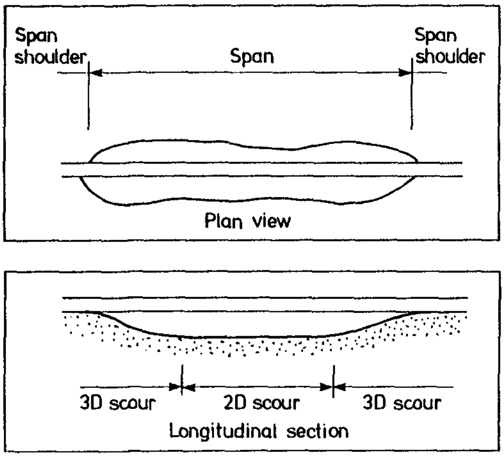

Fig. 8

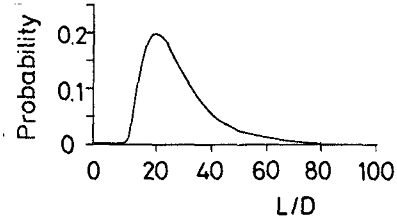

Fig. 9

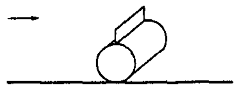

Fig. 10

Effect of pipe sagging on scour

Normally the time scale of span development, T_h, is large compared with T, the time scale of the two-dimensional scour described in the preceding section, (Eq. 10). Here T_h = the time period during which a substantial span length develops to produce some measurable deflection in the middle of the span. The time scales T_h and T may be estimated using the relations given in Hansen et al. (1991) and Fredsøe et al. (1992) (Eq. 12), respectively. While T_h may be in the order of magnitude of days, T is only in the order of magnitude of hours. This means that the pipe normally begins to sag long after the two-dimensional scour has attained its equilibrium stage. However, the sagging may influence the final scour profile and indeed the final scour depth. Effect of pipe sagging on scour has been investigated by various authors, employing two-dimensional laboratory models (Leeuwenstein et al. (1985), Fredsøe et al. (1988), Gökçe & Günbak (1991)). Backfilling and self-burial in the free-span areas has also been investigated by employing the same two-dimensional models in some of these studies.

Length of free spans

The length of free spans is also of engineering interest. Fig. 9 shows the probability density function of span length obtained by Orgill et al. (1992), based on some limited field data which, the authors point out, is very site-specific. The figure shows that there is considerable variation in L/D, ranging from 10 to as much as 100 with the mean value of about 20.

Various authors have considered various factors with regard to the assessment of scour-induced free-span lengths (Fredsøe et al. (1988), Brushi et al. (1986), Eide et al. (1988), Bernetti et al. (1990), Bruschi et al. (1991) and Bijker et al. (1992)). In Fredsøe et al. (1988) work, the free-span length has been related to the so-called stiffness-length of the pipe.

Pipe sinking at span shoulders

Pipe sinking is caused by soil failure, as has already been pointed out in the preceding paragraphs. The soil failure itself may be due to a general shear failure or it may be due to liquefaction.

Sumer & Fredsøe (1993) made an investigation of sinking of pipelines at span shoulders employing a rigid pipe in the laboratory in a current flume. Sumer & Fredsøe's investigation was directed towards the understanding of the sinking process of a pipeline in the soil caused by a general shear failure. A simple formula adopted from soil mechanics regarding the bearing capacity of soil was found to give a satisfactory result in relation to the sinking of the pipe at the span shoulder.

The subject liquefaction in relation to pipelines laid on the sea-bed has been investigated by Silvis (1990). Bijker et al. (1991) gives a brief account of the liquefaction potential of seabed with regard to the presence of laid pipelines.

Stimulated self-burial

As has been seen in the preceding paragraphs, pipelines laid on the seabed may bury themselves in the seabed by various mechanisms. This may occur in the free-span areas where the pipe sags into its scour hole and is covered by sand upon the termination of the sagging process. It may also occur at the span shoulders where the pipe sinks in the sand.

After the discovery that a few months after a 12" pipeline (laid in 1980 on the North Sea bottom in the Dutch sector) caused the formation of a trench, 3 pipe diameter deep and then it gradually sank in this trench and buried itself with the thickness of the covering sand layer of more than 30 cm (Kroezen et al. (1982)), the Delft Hydraulic Laboratory has launched an extensive series of research to investigate the feasibility of a method, called stimulated self-burial, to exploit the self-burial potential (Hulsbergen (1984), Hulsbergen (1986), Hulsbergen & Bijker (1989)). This is particularly important in the circumstances where pipelines would not bury themselves fast enough or they would not at all bury themselves. The idea is to stimulate a controlled local scour by using fins (called spoiler) attached to the pipeline as sketched in Fig. 10. From the research carried out by the Delft Hydraulics, it was found that, firstly, the spoiler reduces the time necessary to accomplish a given embedment level by a factor of ten with respect to a plain pipe, and secondly, the final depth of burial is larger than in the case of plain pipelines. The same effects have been observed also in waves (Gökçe & Günbak (1991)).

4. EFFECT OF SCOUR ON FORCES AND VIBRATIONS

Obviously forces on and vibrations of a pipe over a scoured bed will be different from those experienced when the pipe sits on a plane bed. Jensen et al. (1990) has investigated forces on a pipe at different stages of scour process in steady current, using frozen scoured-bed models. Stansby & Starr (1991) has measured forces on a pipe gradually sinking in the bed under flow action in both steady current and waves.

Effect of scour on hydroelastic vibrations of pipelines, on the other hand, has been studied in steady currents (Sumer et al. (1988a), Kristiansen (1988) and Kristiansen & Tørum (1989)) and in waves (Sumer et al. (1989)).

Forces have been measured also on pipeline models with attached spoilers with different embedment ratios (representing different stages of the self-burial process) (Hulsbergen & Bijker (1989)).

5. MATHEMATICAL MODELLING

The studies concerning mathematical modelling of scouring process may broadly be divided into three categories: 1) the studies dealing with mathematical models based on potential-flow theory, 2) those dealing with k-ε models and 3) those dealing with discrete-vortex models.

Regarding the potential-flow models, Chao & Hennessy (1972) was the first to apply the simple potential solution to the flow in the gap between the pipe and the bed. As a matter of fact, the potential flow around a cylinder placed near a wall was described by von Müller in as early as 1929. Müller's potential flow description was modified by Fredsøe & Hansen (1987) by taking into consideration the actual, measured velocity at the top and bottom edges of the cylinder. This modification is significant particularly in the case of very small gaps where the potential theory overpredicts the velocity in the gap. Hansen et al. (1986) later implemented the same line of thought as in Fredsøe & Hansen, and developed a potential theory for the case of a cylinder over a *scoured* bed. In both models, only the flow upstream of the pipe is described by the potential theory. (Clearly the downstream flow, i.e. the lee-wake, cannot be described by a potential flow model). The results of scour prediction by Hansen et al.'s (1986) potential flow description agreed satisfactorily with the experiments. Recently, Hansen (1992), with a simple representation of the lee wake, extended this model so as to cover waves. His model results give a fairly good agreement with Sumer & Fredsøe's (1990) empirical result (Eq. 9). Using the same model, he also examined the effect of Shields parameter, the effect of current in combined waves and current and the effect of angle of attack. Bernetti et al. (1990) has also developed a mathematical model of scour below pipelines. The model has two main components, namely the two-dimensional scour component and the three-dimensional scour component. The two-dimensional scour component was based on Chao & Hennessy's (1972) potential-flow description. Bernetti et al.'s model is able to predict also the development of the three-dimensional scour, the free-span length and sinking of the pipe at span shoulders. The model has been tested against simple cases where data is available (Mao (1986)). The model performance for these test cases appeared to be quite satisfactory.

The second group of mathematical modelling studies concerns the k-ε simulation of the flow (Leeuwenstein & Wind (1984) and van Beek & Wind (1990)). As is well known, the k-ε model is successfully used in various fields of fluid mechanics (Rodi (1984)). Regarding its application in scour below pipelines, Leeuwenstein & Wind has calculated the flow around the pipe over a scoured bed with such a model and made the morphological calculations by use of sediment continuity equation along with a sediment-transport equation. In the follow-up study, van Beek & Wind extended the model so as to cover also the suspended-load transport in the morphological calculations.

The third group of studies in mathematical modelling of scour concerns the discrete-vortex-model simulation of the flow around pipes (Sumer et al. (1988a), Jensen et al. (1990) and Jensen et al. (1989)). The particular model which has been used in these studies is the so-called cloud-in-cell method. The computational details of the method is well documented in the literature (see for example Stansby & Dixon (1983)). The method is able to predict the gross behaviour of the vortices in the lee wake of the pipe. This enabled Sumer et al. (1988b) to study the effect of lee-wake vortices on the bed shear stress downstream of the pipe over both a plane bed and a scoured bed. The method was later used by the same group to simulate flow around a pipe over a scoured bed in steady currents (Jensen et al. (1990) and in waves (Jensen et al. (1989)).

Finally, it may be mentioned that an integrated approach has been adopted recently by the Danish Hydraulic Institute and the Delft Hydraulics in a joint study, to develop a computer model which would enable the engineer to decide on occurrence and disappearance of scour along a pipeline/cable, pipeline/cable self-burial, trench backfilling, migrating sand waves exposure and undermining of pipelines or cables. A brief account of this study has been reported by Staub & Bijker (1990).

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