Bottom Deformation of Dock Settling Basin on Elastic Foundation

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Abstract: This research studies the deformation computation of bottom of the dock settling basin mainly during the operation period. The bottom of dock settling basin along its length is supposed as a finite bending rigid beam on the ground base. The Fuss-Winkler model is used to compute the variable stiffness coefficient. The basic parameter of this model, i.e. Soil stiffness coefficient, is a non-linear parabolic equation that is accepted along the length of the dock settling basin bottom. This problem is shortened to variable coefficient of ordinary differential equation and is computed by boundary conditions and Maclaurin's series method. After computation of equation, deformation rate and interior forces can be found at any arbitrary cross section of dock settling basin bottom. This article presents a new method of computing the rate and quality of the dock settling basin bottom deformation. The results are compared to the results of the some other researchers.

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1. Introduction

Throughout different stages of operation, settling basin structures are subjected to varying loads over very short periods of time. Specifically, rapid changes in loaded can occur during settling basin flooding or dewatering. There are three phases of settling basin operation during which certain specific external loads and load combinations are acting on the settling basin structure: Phase 1: The empty settling basin is dewatering during periods of scouring (Cleaning). During this period the settling basin walls may be loaded by backfill soil, water pressures and by the reaction of the settling basin bottom. The settling basin bottom is loaded by hydrostatic uplift, the reaction of the sub-grade and walls. Phase 2: The settling basin is dewatering but with silt sediments on the settling basin bottom. The external loads may be the same as in Phase 1, but the weight of the silt sediments must be added. Phase 3: the empty settling basin is flooded. The hydrostatic pressures on the settling basin bottom and settling basin walls are added to loads occurring in Phase 1.

Furthermore, in all three operational phases loads from the relevant structure and/or operational mechanical equipment must be added to the aforementioned external loads. The weight of the empty settling basin structure and soil estimated and used in all computing process according to generally accepted methods. The hydrostatic pressure acting on settling basin walls, as well as hydrostatic uplift, depends on the water levels around the settling basin structure and the permeability of the settling basin foundation. For computing the hydrostatic pressures and the uplift load acting on the settling basin structure, the highest possible water level of the surrounding ground observed over a long period of time is considered. If the base of the settling basin can reach strata in which the groundwater in under pressure, the hydrostatic uplift load is calculated for highest possible static water level. Lateral soil pressure on the settling basin walls are estimated by conventional methods. The type of structure used for the side walls has a major impact on the soil lateral thrust; for example, massive side walls and particularly those rigidly joined with the foundation slab will be exposed to soil pressure "at rest" while flexible walls, or walls that allow for certain movement, will be exposed to active soil pressures.

It is assumed that the dock k settling basin was completely filled with water during the First limit exploitation. There is silt to a specific volume of the dock and the height of the backfill is same as the water height in the dock. In this case the wall and bottom structure of dock settling basin are affected by the forces that were created by different influences. Bottom structure of dock settling basin is affected by hydrostatic pressure of the dock, settled silts weight force and effects on the wall including (hydrostatic pressure, weight force of settling silts, active pressure of backfill, the concrete wall weight force etc.) and the bottom structure is affected by vertical and shearing forces at the initial and ending cross section, pile moment and axial force. Main effects on settling basin cell are shown in figures (1) and (2). Deformation scheme of settling basin bottom structure in this case is shown in figure (3).

Computation of deformation is surveyed by different researchers. C.N. Klepikov (1967) proposed the computation method of deformation of dock settling flexible bottom structure of ship entering sluice. This method, by dividing the non-linear curve of sub-grade stiffness coefficient along the bottom structure length into the special elements, accepts the curve of the each element linear. In general by apply-

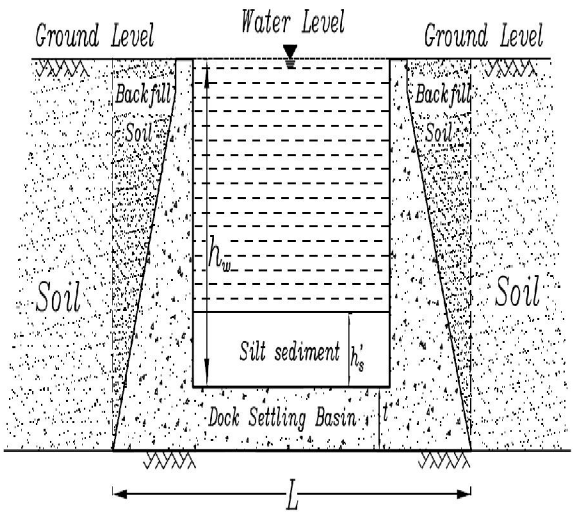

Figure 1. Dock settling basin scheme

ing ultimate element method in finite number the problem is solved to system equation. The solution is generated by using the initial parameters.

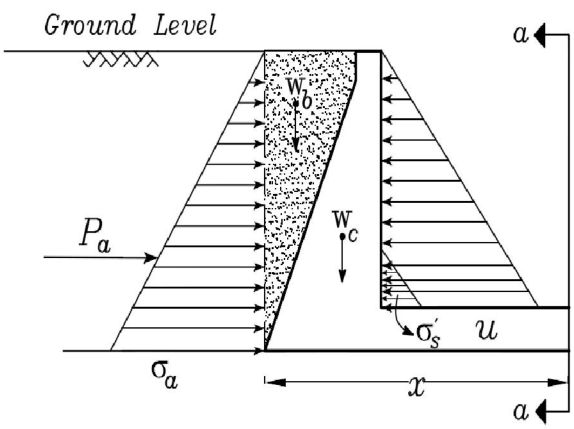

Figure 2. Diagram of forces effect on the dock settling basin

B.A. Kocitcyn (1971) when computing the flexible foundation beams accepted the sub-grade stiffness coefficient non-linear. Foundation stiffness coefficient curve, dependent on the deflection of foundation is accepted convex or concave parabolic. The problem is solved by fourth order differential equation and special boundary conditions. E.A. Simvulidi (1978) offered the deformation computation method of elastic foundation for engineering structures. In this method the reactive resistance of sub-grade is computed by fourth order polynomial. All of the loads affecting the beam, according to shear function theorem, are substituted for uniformly distributed load and each problem is computed by the fourth order differential equation. The differential soil-structure interaction problems are solved in here. The author in this method mainly used the elastic half-space theory. M.J. Qorbunov–Posadov and others (1984) had done the computation method of flexible beam elements in elastic sub-grade. The author mainly used the elastic half-space theorem. This space generally characterized with the deformation model and Poison coefficient. The author solved the contact problem by accepting the sub-grade reactive resistance as eighth order polynomial. K.M. Mammadov (2008) performed experiments about deformation computation of flexible sub-grade beams of different geometric forms in finite confined layer. In this method the flexible sub-grade beams with variable stiffness coefficient in the sub-grade is generally solved by fourth order differential equation and infinite series method are mainly used for the solution.

2. Definiation of Contact Problem

Bottom structure ending transverse sections correspond are affected by the same forces. Bottom structure along its length is affected by the q distributed load and at the initial transverse section, by the M_o pile moment, Q_o Shearing force, N axial force. Bottom structure is considered as a beam with constant bending stiffness beam on the flexible ground base. Beam Width is considered one meter (in the condition of flatness equation). The bottom structure is supposed as flexible beam because of the bending and tension forces. As the bottom structure is bended by these effects, ground basin reflects reactive resistance against this bending. Fuss-Winkler variable stiffness coefficient model is used to determine the intensity of resistance:

$$q_{qr}(x) = -K(x) \cdot Y(x), \qquad (1)$$

where

K(x) is the variable stiffness coefficient along the foundation length.

Y(x) is the bending deformation of the structure at the arbitrary transverse section.

According to the researches carried out by K.M. Mammadov, stiffness coefficient change of structure uniformly-distributed loaded foundation along its length is accepted as a three term quadratic parabola form:

$$K(x) = k_o - \frac{4(K_o - K_c)}{L}x + \frac{4(K_o - K_c)}{L^2}x^2 \qquad (2)$$

where

K_o is the stiffness coefficient of initial transverse section of bottom structure ground basin;

K_s is the stiffness coefficient of ground basin at the middle of the beam;

$K_l = K_o$ is the foundation ground stiffness coefficient at the right end transverse section of bottom structure.

In order to compute the tension-bending differential equation of bottom structure we write the bending moment equation at the arbitrary transverse section:

$$M(x) = M_o - Q_o \cdot x - q \cdot \frac{x^2}{2} + N\left[Y(x) - Y_o\right] + M_{qr}(x) \quad (3)$$

$M_{qr}(x)$ is the developed moment of ground resistance intensity at the bottom structure arbitrary transverse section. According to the differential relation between deflection and bending moment we write:

$$\mathrm{E} \cdot J \frac{d^2 Y(x)}{dx^2} = -M(x) = -M_o + Q_o \cdot x + q\frac{x^2}{2} - N\left[Y(x) - Y_o\right] - M_{qr}(x) \quad (4)$$

If we differentiate equation (4) again with respect to (x), so:

$$\mathrm{E} \cdot J \frac{d^3 Y(x)}{dx^3} = Q_o + qx - NY'(x) - Q_{qr}(x) \quad (5)$$

According to (4) and (5) equations, E.J is the bottom structure constant stiffness coefficient, and $Q_{qr}(x)$ is the developed shearing force of the ground reactive resistance at the arbitrary transverse section. If we differentiate again equation (5) with respect to (x):

$$\mathrm{E} \cdot J \frac{d^4 Y(x)}{dx^4} = P(x) = q - NY''(x) - q_{qr}(x)$$

(6)

where

$P(x)$ is the intensity of the uniformly-distributed load.

If we consider (1) and (2) equations, we can determine $q_{qr}(x)$ by the following formulas:

$$q_{qr}(x) = -\left[K_o - \frac{4(K_o - K_s)}{L}x + \frac{4(K_o - K_s)}{L^2}x^2\right]Y(x) \quad (7)$$

If we consider equation (7) in equation (6) and divide all terms into E.J:

$$Y^{IV}(x) = \bar{q} - \upsilon^2 Y''(x) - \left[\alpha_o - \alpha_1 x + \alpha_2 x^2\right]Y(x) \quad (8)$$

The following conditional denotation is accepted:

$$\bar{q} = \frac{q}{\mathrm{E} \cdot J}\left[m^{-3}\right];\ \upsilon^2 = \frac{N}{\mathrm{E} \cdot J}\left[m^{-2}\right];\ \alpha_o = \frac{K_o}{\mathrm{E} \cdot J}\left[m^{-4}\right];$$
$$\alpha_1 = \frac{4(K_o - K_s)}{L \cdot \mathrm{E} \cdot J}\left[m^{-5}\right];\ \alpha_2 = \frac{4(K_o - K_s)}{L^2 \cdot \mathrm{E} \cdot J},\left[m^{-6}\right] \quad (9)$$

We consider differential equation (8) at the bottom structure left initial transverse section by the following boundary conditions:

$$Y(0) = Y_o;\ Y'(0) = \theta_o;\ Y''(0) = \frac{M_o}{\mathrm{E} \cdot J} = \overline{M}_o;$$
$$Y'''(0) = \frac{Q_o}{E \cdot J} - \upsilon^2\theta_o = \overline{Q}_o - \upsilon^2\theta_o \quad (10)$$

In the last equation deflection (Y_o), slope (rotate angle)(θ_o), bending moment (M_o) and shearing force (Q_o) are the initial parameters at the beginning of bottom structure. (8) equation is an ordinary fourth order differential equation. It can't be solved in quadrature. Different approximate methods are used to solve this equation. The principle methods as the variation methods of structures mechanics, A.N. Krilov numerical computation method, Series method, Picard limit of a sequence method, etc [1,2,3,4,5,6,7] are used in the computation of this equation. Series method is used in the computation of (8) equation.

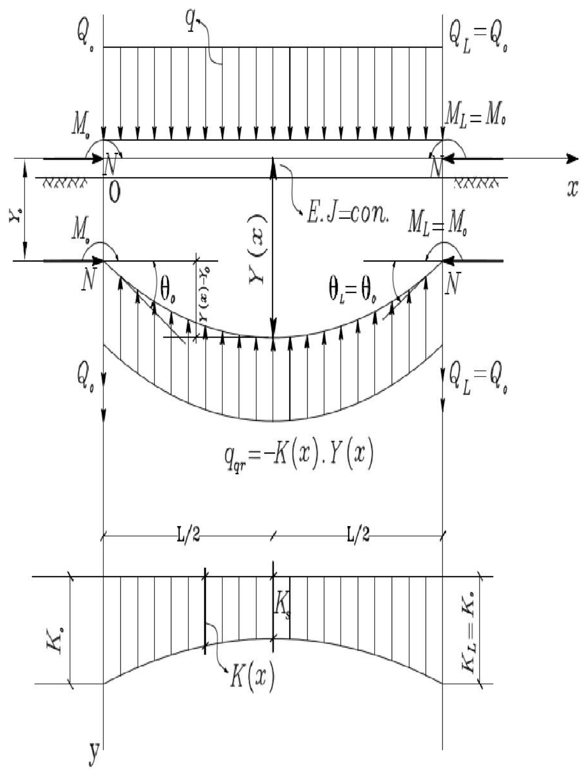

Figure 3. Deformation scheme of dock-kind settling basin bottom structure

3. Method of Solution

It is very important to use the Series method to compute the (8) equation. If we compute the y(x) function in the form of the Maclaurin's series, so we write:

$$Y(x) = Y(0) + Y^{/}(0)\frac{x}{1!} + Y^{//}(0)\frac{x^2}{2!} + Y^{///}(0)\frac{x^3}{3!} + Y^{/V}(0)\frac{x^4}{4!} + ... + Y^{(n)}(0)\frac{x^n}{n!} + ... \quad (11)$$

The zero values of Y(0), $Y^{/}(0)$, $Y^{//}(0)$, $Y^{///}(0)$ are accepted as initial conditions of (8) equation. Four order derivative value of Y(x) function more than Foru-order, by considering the initial boundary conditions (8) equation, can be computed by consequence differentiating of the (6) differential equation. If we substitute the zero value of all of the derivatives in Maclaurin's series and we qroup the gained functions according to the four beginning parameters and intensity of uniformly distributed load then we can solve the (8) equation as follow:

$$\begin{cases} Y_n(x) = Y_o F_1(x) + \theta_o F_2(x) - \dfrac{M_o}{E.J} F_3(x) \\ + \dfrac{Q_o}{E.J} F_4(x) + \dfrac{q}{E.J} F_5(x) \\ \theta(x) = Y_o F_1^{/}(x) + \theta_o F_2^{/}(x) - \overline{M}_o F_3^{/}(x) \\ + \overline{Q}_o F_4^{/}(x) + \overline{q} F_5^{/}(x) \\ \dfrac{M_n(x)}{E.J} = Y_o F_1^{//}(x) + \theta_o F_2^{//}(x) \\ - \overline{M}_o F_3^{//}(x) + \overline{Q}_o F_4^{//}(x) + \overline{q} F_5^{//}(x) \\ \dfrac{Q^{Lang.}(x)}{E.J} = Y_o \left[F_1^{///}(x) - v^2 F_1^{/}(x) \right] \\ + \theta_0 \left[F_2^{//}(x) - v^2 F_2^{/}(x) \right] - \overline{M}_o \left[F_3^{//}(x) - v^2 F_3^{/}(x) \right] \\ + \overline{Q}_o \left[F_4^{//}(x) - v^2 F_4^{/}(x) \right] + \overline{q} \left[F_5^{///}(x) - v^2 F_5^{/}(x) \right] \end{cases} \quad (12)$$

$F_j(x)$ functions in equations (12) are considered as a rapid converging series sum. In practical computations they might be satisfied by the first two or three continuous series at the right side of the $F_j(x)$ functions and also the first two or three terms of each series. Convergence series are formed by the small values of parameters at the numerator of each series and also increasable factorial value at the denominator of each series. Approximate derivations of $F_j(x)$ function at the last three equations of equ-ations (12) can be specified by sequential differentiating with respect to *(x)* in these functions. Generally the solution of (12) equations, $F_1(x)$, $F_2(x)$, $F_3(x)$, $F_4(x)$ *and* $F_5(x)$ functions are the four independent special solution, homogeneous independent, of (8) differential equation. $F_5(x)$ function is a heterogeneous special solution of the (8) differential equation. So the unknown functions are specified by using the continuous and rapid converging series. General solution of the contact problem is presented by the initial parameters. In this way it is possible to find the values of unknown parameters easily. According to the formed solution let's consider a special case. It is supposed that the stiffness coefficient of dock kind settling ground basin has an integral mean value: i.e.

$$K_{ave.} = \frac{1}{L}\int_0^l k(x)dx = \frac{1}{L}\int_0^l \left[k_0 - \frac{4(k_0 - k_s)}{L}x + \frac{4(k_0 - k_s)}{L^2}x^2 \right] dx = \frac{(k_0 + 2k_s)}{3} \quad (13)$$

So it is clear that:

$k_o = k_c = k_l = k_{ave.} \Rightarrow \alpha_0 = \alpha_{ave.}$ & $\alpha_1 = \alpha_2 = 0$,

And finally $F_j(x)$ functions can be written as follow:

$$\begin{cases} F_1(x) = 1 + \sum_{n=1}^{\infty} (-1)^n \dfrac{\alpha_{ave.}^n \cdot x^{4n}}{(4n)!} + \\ \dfrac{\alpha_{ave.}}{v^4}(1 - \dfrac{v^2x^2}{2!} + \dfrac{v^4x^4}{4!} - \cos vx) + \ldots \\ F_2(x) = x + \sum_{n=1}^{\infty} (-1)^n \dfrac{\alpha_{ave.}^n \cdot x^{4n+1}}{(4n+1)!} \\ + \alpha_{ave.} \sum_{n=1}^{\infty} (-1)^{n+1} \dfrac{v^{2n} \cdot x^{2n+5}}{(2n+5)!}(n+1) + \ldots; \\ F_3(x) = \dfrac{x^2}{2!} + \sum_{n=1}^{\infty} (-1)^n \dfrac{\alpha_{ave.}^n \cdot x^{4n+2}}{(4n+2)!} \\ - \dfrac{1}{v^2}(1 + \dfrac{v^2x^2}{21} - \cosh vx) \\ + \alpha_{ave.} \sum_{n=1}^{\infty} (-1)^{n+1} \dfrac{v^{2n} \cdot x^{2n+6}}{(2n+6)!}(n+1) + \ldots \\ F_4(x) = \dfrac{x^3}{3!} + \sum_{n=1}^{\infty} (-1)^n \dfrac{\alpha_{ave.}^n \cdot x^{4n+3}}{(4n+3)!} \\ + \dfrac{\alpha_{ave.}}{v^3}(vx - \dfrac{v^3x^3}{3!} - \sinh vx) \\ + \alpha_{ave.} \sum_{n=1}^{\infty} (-1)^{n+1} \dfrac{v^{2n} \cdot x^{2n+7}}{(2n+7)!}(n+1) + \ldots \\ F_5(x) = \dfrac{x^4}{4!} + \sum_{n=1}^{\infty} (-1)^n \dfrac{\alpha_{ave.}^n \cdot x^{4n+4}}{(4n+4)!} \\ - \dfrac{1}{v^4}(1 - \dfrac{v^2x^2}{2!} + \dfrac{v^4x^4}{4!} - \cosh vx) \\ + \alpha_{ave.} \sum_{n=1}^{\infty} (-1)^{n+1} \dfrac{v^{2n} \cdot x^{2n+8}}{(2n+8)!}(n+1) + \ldots). \end{cases} \quad (14)$$

If the effect of the tension force in the last equation is not considered, i.e. it is accepted that $\upsilon^2 = \dfrac{N}{E \cdot J} = 0$, (14) functions are written in the following form:

$$\begin{cases} F_1(x) = 1 + \sum_{n=1}^{\infty} (-1)^n \frac{\alpha_{ave.}^{\ n} \cdot x^{4n}}{(4n)!} \\ F_2(x) = x + \sum_{n=1}^{\infty} (-1)^n \frac{\alpha_{ave.}^{\ n} \cdot x^{4n+1}}{(4n+1)!} \\ F_3(x) = \frac{x^2}{2!} + \sum_{n=1}^{\infty} (-1)^n \frac{\alpha_{ave.}^{\ n} \cdot x^{4n+2}}{(4n+2)!} \\ F_4(x) = \frac{x^4}{3!} + \sum_{n=1}^{\infty} (-1)^n \cdot \frac{\alpha_{ave.}^{\ n} \cdot x^{4n+3}}{(4n+3)!} \\ F_5(x) = \frac{x^4}{4!} + \sum_{n=1}^{\infty} (-1)^n \cdot \frac{\alpha_{ave.}^{\ n} \cdot x^{4n+4}}{(4n+4)!} \end{cases} \tag{15}$$

Equation (15) is the separation of A.N. Krilov hyperbola-trigonometric function into infinite series. If it is supposed that:

$$\alpha = \sqrt[4]{\frac{\alpha_{ave.}}{4}} \ or \ \alpha_{ave.} = 4\alpha^4 \tag{16}$$

The complete solution of equation (8) can be represented as follow:

$$\begin{cases} \overline{F}_1(\alpha x) = \cosh\alpha x \cdot \cos\alpha x \\ \overline{F}_2(\alpha x) = \frac{1}{2}(\sinh\alpha x \cdot \cos\alpha x + \cosh\alpha x \cdot \sin\alpha x); \\ \overline{F}_3(\alpha x) = \frac{1}{2}(\sinh\alpha x \cdot \sin\alpha x); \\ \overline{F}_4(\alpha x) = \frac{1}{4}(\cosh\alpha x \cdot \sin\alpha x - \sinh\alpha x \cdot \cos\alpha x); \\ \overline{F}_5(\alpha x) = \frac{1}{4}(\cosh\alpha x \cdot \cos\alpha x) = \frac{1}{4}\overline{F}_1(\alpha x) \end{cases} \tag{17}$$

Four initial parameters are included in general solution of equations (12). M_o and Q_o are considered as known parameters. The next two Y_o and θ_o kinematic initial parameters are specified by the boundary conditions at the right corner of dock kind settling bottom structure. These boundary conditions can be written as follow [6]:

$$\frac{M_n(L)}{E \cdot J} = \frac{M_L}{E \cdot J}, \quad \frac{Q_n^{leng.}(L)}{E \cdot J} = \frac{Q_L}{E \cdot J} \tag{18}$$

According to the last two conditions of equations (12) and by using these conditions we can find:

$$\begin{cases} Y_o \cdot F_1''(L) + \theta_o \cdot F_2''(L) = \alpha_1(L) \\ Y_o \cdot \alpha_2(L) + \theta_o \cdot \alpha_3(L) = \alpha_4(L), \end{cases} \tag{19}$$

Where

$$\begin{cases} \overline{M}_o\left[1 - F_3''(L)\right] - \overline{Q}_o \cdot F_4''(L) - \overline{q} \cdot F_5''(L) = \alpha_1(L) \\ F_1'''(L) - \upsilon^2 \cdot F_1'(L) = \alpha_2(L) \\ F_2'''(L) - \upsilon^2 \cdot F_1'(L) = \alpha_3(L) \\ \overline{M}_o\left|\upsilon^2 F_3'(L) - F_3'''(L)\right| \\ + \overline{Q}_o\left|1 - F_4'''(L) + \upsilon^2 F_4'(L)\right| \\ - \overline{q}\left|F_5'''(L) - \upsilon^2 F_5'(L)\right| = \alpha_4(L) \end{cases} \tag{20}$$

4. Conclusion

By this method we can compute the deformation of dock settling basin of bottom structure as a flexible beam on the basin ground and also compute the created bending deflection, angular deflection, bending moment and shearing force at the arbitrary transverse section of the bottom structure.

Different researchers considered the different values of bending moments. Those values are compared with the values of bending moment of presented method along the beam lengths (10 m). In figure 4, maximum bending moment value gained by M.J. Qorbunov-Posadov method and minimum value gained by B.A. Kocitcyn method. The maximum bending moment for a simply supported beam under a uniformly distributed load (present study) in comparison to the K.M. Mammadov method is decreasing by (6.04%). In figure 5, maximum shearing force values gained by M.J. Qorbunov-Posadov method and minimum value gained by B.A. Kocitcyn method. The maximum shearing force for a simply supported beam under a uniformly distributed load (present study) in comparison to the M.J. Qorbunov-Posadov method is decreasing by (5.49%).

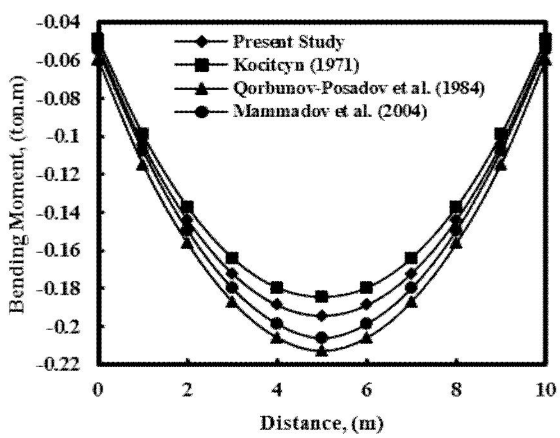

Figure 4. Bending moment curves for a simply supported under a uniformly distributed load

Finally in the offered method, Bending moment and shearing force curves have accordance with the investigation of C.N. Klepikov, B.A.

Kocitcyn, M.J. Qorbunov-Posadov and K.M. Mammadov, but solution of the problem is completely different from those methods.

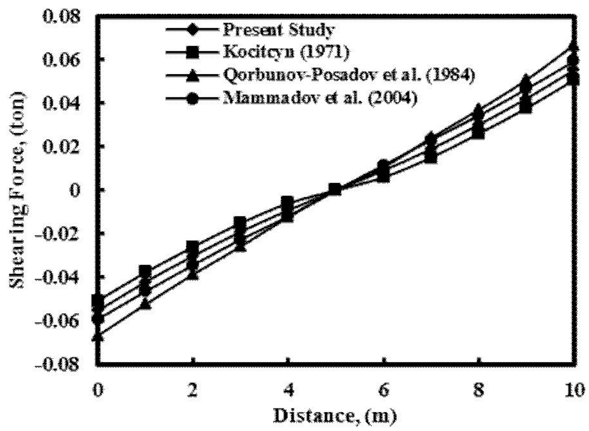

Figure 5. Shearing force curves for a simply supported under a uniformly distributed load

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