Analysis of beam on elastic foundation using the radial point interpolation method

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Abstract The concept of beam on an elastic foundation has been extensively used by geotechnical engineers for foundation design and analysis. However, most numerical solutions for beam on an elastic foundation are obtained by mesh based methods, such as finite element or finite difference methods. Mesh based methods suffer from some deficiencies, mostly related to mesh definition. In this paper, a mesh-free method, called the radial point interpolation method, is implemented for the analysis of a beam on two parameter elastic foundation. The beam and the elastic foundation are modeled separately. The geometry of the beam is simulated by a set of nodes that are aligned on two or three parallel lines. The displacement field along the beam is constructed by radial basis functions, and the discretized system of equations is derived by substitution of the displacement field into the weak form of the governing equation. The elastic foundation is simulated by the concept of the linkage element and there is no need for nodes or elements in the traditional sense. The stiffness of the foundation has been taken into account by defining normal and tangential stiffness coefficients along the foundation layer. The displacement of each point across the foundation layer is tied to the displacement of the beam nodes. The final system of equations is derived by a combination of equations for both the beam and the elastic foundation in the global coordinate. Based on the derived equations, a computer code has been developed and the results of analysis with the mesh-free method are compared with the results of the exact solution and results obtained from finite element analysis.

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1. Introduction

There are many geotechnical engineering problems that can be idealized as beams on elastic foundations. This kind of modelling helps to understand the soil-structure interaction phenomenon and predict the contact pressure distribution and deformation within the medium. The most common theory for a beam on elastic foundation modelling is the Winkler approach [1]. However, the modelling of soil using the Winkler theory was considered inadequate in the handling of various problems. The main weakness lies in the fact that it overlooks the shear interaction between the spring elements. Hence, some other models were proposed some decades ago [2–6], and also in recent studies [7,8]. These models have been introduced on the refinement of the Winkler approach by application of various types of interconnection between Winkler springs. One of the simplest forms of two-parameter elastic foundation models is the model with both normal and tangential stiffness along the elastic foundation [2]. The numerical solution for these two-parameter models are mainly obtained by mesh based methods, such as Finite Element (FEM) or Finite Difference Methods (FDM) [9]. Although the mesh based methods are robust, and widely used, they suffer from some deficiencies, which are mostly related to mesh definition. Hence, the author has been encouraged to implement a new class of numerical methods, which are globally coined as mesh-free or mesh-less methods, for analysis of beams on an elastic foundation.

Mesh-Free Methods (MFM) were developed by Lucy [10], who introduced Smoothed Particle Hydrodynamics (SPH) for modeling astrophysical phenomena. Libersky and Petschek [11] extended this method to solve the solid mechanics problem.
Nayroles et al. [12] extended a new branch of MFM by using a basis function and a weight function to form a local approximation based on a set of nodes. Belytschko et al. [13] improved this method by introducing the moving least square approximation and called their method "element free Galerkin". Many other researchers proposed various MFM, such as finite cloud [14], the reproducing kernel particle method [15], and the point interpolation method [16], etc.. Their main characteristic is that there is no need for mesh in the traditional sense.

In this paper, the enriched radial point interpolation method is implemented for the analysis of a beam on an elastic foundation. Based on the derived equations, a computer code has been developed and its validity investigated by solving some examples at the end of the paper.

2. Enriched radial point interpolation method

Polynomials have been used as basis functions in interpolation to create shape functions in many numerical methods, such as FEM. In the FEM, however, the interpolation is based on elements that have no gap and overlapping. In the point interpolation method (PIM), interpolation is based on a small set of nodes in the vicinity of a desired point, named the local support domain (Figure 1). Support domains of different points can overlap each other.

Consider a continuous function, \( u(\mathbf{X}) \) (i.e. displacement function). This function can be approximated in the vicinity of \( \mathbf{X} \) as follows [18]:\

\[
 u(\mathbf{X}) = \sum_{i=1}^{n} p_i(\mathbf{X})a_i = \mathbf{P}^T \mathbf{a},
\]

where \( p_i(\mathbf{X}) \) is the polynomial basis function of spatial coordinate \( \mathbf{X} = [x, y]^T \), \( n \) is the number of nodes in the support domain of \( \mathbf{X} \), and \( a_i \) is the corresponding coefficient of the basis function. In the matrix form, shown in Eq. (1),\

\[
 \mathbf{P} = \begin{bmatrix} p_1(\mathbf{X}) & p_2(\mathbf{X}) & \cdots & p_n(\mathbf{X}) \end{bmatrix}^T,
\]

\[
 \mathbf{a} = \begin{bmatrix} a_1 & a_2 & \cdots & a_n \end{bmatrix}^T,
\]

the unknown coefficient, \( a_i \), can be determined by enforcing \( u(\mathbf{X}) \) to be the nodal displacement at \( n \) nodes in the support domain. It can then be written as:\n
\[
 \mathbf{U}_5 = \mathbf{P} \mathbf{a},
\]

where \( \mathbf{U}_5 \) is the vector of nodal displacements,\

\[
 \mathbf{U}_5 = [u_1 \quad u_2 \quad u_3 \quad \cdots \quad u_n]^T,
\]

and \( \mathbf{P} \) is the polynomial moment matrix.\

\[
 \mathbf{P}_Q = \begin{bmatrix} 1 & x_1 & y_1 & x_1y_1 & \cdots & p_n(X_1) \\ 1 & x_2 & y_2 & x_2y_2 & \cdots & p_n(X_2) \\ \vdots & \vdots & \vdots & \vdots & \ddots & \vdots \\ 1 & x_n & y_n & x_ny_n & \cdots & p_n(X_n) \end{bmatrix}
\]

Assuming the existence of \( \mathbf{P}_Q^{-1} \), a unique solution of vector \( \mathbf{a} \) can be obtained as:\n
\[
 \mathbf{a} = \mathbf{P}_Q^{-1} \mathbf{U}_5.
\]

Substituting Eq. (7) into Eq. (1) yields:\n
\[
 u(\mathbf{X}) = \mathbf{P}^T(\mathbf{X}) \mathbf{P}_Q^{-1} \mathbf{U}_5 = \sum_{i=1}^{n} \psi_i u_i = \mathbf{\Phi}^T(\mathbf{X}) \mathbf{U}_5,
\]

where \( \mathbf{\Phi}(\mathbf{X}) \) is the vector of PIM shape functions.\

\[
 \mathbf{\Phi}^T(\mathbf{X}) = [\psi_1(\mathbf{X}) \quad \psi_2(\mathbf{X}) \quad \cdots \quad \psi_n(\mathbf{X})].
\]

The shape functions constructed by PIM have the Kronecker delta function property, which allows the simple imposition of essential boundary conditions, as in conventional FEM.

The PIM is accurate and easy to use. However, an inappropriate choice of polynomial basis terms, or the bad arrangement of nodes in the support domain near \( \mathbf{X} \) will result in a singular matrix, \( \mathbf{P}_Q \). Several strategies have been proposed to overcome this problem [18]. Using Radial Basis Functions (RBF) is one of the best solutions to guarantee the invertability of \( \mathbf{P}_Q \). There are different functions, such as multi-quadratic, Gaussian, and Logarithmic, which can be used as RBF. In this paper, the multi-quadratic form has been used as follows:\n
\[
 R_i(\mathbf{X}) = [r_i + c^2]^q,
\]

where \( r_i \) is the distance between the desired point (\( \mathbf{X} \)) and field node (\( \mathbf{X}_i \)) defined as:\n
\[
 r_i = [(x - x_i)^2 + (y - y_i)^2]^0.5.
\]

c and \( q \) in Eq. (10) are constants having best values that should be determined during the solution, and depend on the type of problem. For the solid mechanic problems, Liu [18] suggests 1.42 and 0.98 for \( c \) and \( q \), respectively. The mentioned values are used in this paper.

To ensure the consistency of the radial point interpolation method (RPIM), polynomial terms are added to pure radial basis
functions. This also improves the accuracy of the results. So, the approximation of function $u(X)$ can be written as [18]:

$$u(X) = \sum_{i=1}^{n} R_i(X) a_i + \sum_{j=1}^{m} p_j(x_i, y_i) b_j = R^T(X) a + P^T(X) b,$$  \hfill (12)

where:

$$R(X) = [R_1(X), R_2(X), R_3(X), \ldots, R_n(X)]^T,$$  \hfill (13)

$$P(X) = [p_1(X), p_2(X), p_3(X), \ldots, p_m(X)]^T,$$  \hfill (14)

$$a = [a_1, a_2, a_3, \ldots, a_n]^T,$$  \hfill (15)

$$b = [b_1, b_2, b_3, \ldots, b_m]^T,$$  \hfill (16)

where $R_i$ and $p_j$ are the radial and polynomial basis function, respectively, $a_i$ and $b_j$ are interpolation coefficients, $m$ is the number of polynomial terms and $n$ is the number of nodes in the support domain of $X$. For $a_i$ and $b_j$ determination, $m + n$ equations are needed. The $n$ equations can be produced by enforcing $u(X)$ to be the nodal displacement at $n$ nodes in the support domain. So, we have:

$$u_k = u(x_k, y_k) = \sum_{i=1}^{n} R_i(x_k, y_k) a_i + \sum_{j=1}^{m} p_j(x_k, y_k) b_j$$

$$k = 1, 2, \ldots, n,$$  \hfill (17)

or in matrix form:

$$U_5 = R_0 a + P_0 b,$$  \hfill (18)

where:

$$R_0 = \begin{bmatrix} R_1(r_1) & R_2(r_1) & \cdots & R_n(r_1) \\ R_1(r_2) & R_2(r_2) & \cdots & R_n(r_2) \\ \vdots & \vdots & \ddots & \vdots \\ R_1(r_n) & R_2(r_n) & \cdots & R_n(r_n) \end{bmatrix},$$  \hfill (19)

$$P_0 = \begin{bmatrix} p_1(x_1, y_1) & p_2(x_1, y_1) & \cdots & p_m(x_1, y_1) \\ p_1(x_2, y_2) & p_2(x_2, y_2) & \cdots & p_m(x_2, y_2) \\ \vdots & \vdots & \ddots & \vdots \\ p_1(x_n, y_n) & p_2(x_n, y_n) & \cdots & p_m(x_n, y_n) \end{bmatrix}_{n \times m}.$$  \hfill (20)

$m$ remaining equations can be gained through:

$$\sum_{i=1}^{n} p_j(x_i, y_i) a_i = 0 \quad j = 1, 2, \ldots, m,$$  \hfill (21)

which is the uniqueness condition for solutions.

In matrix form, Eq. (21) can be written as:

$$P^T a = 0.$$  \hfill (22)

Combining Eqs. (18) and (22) gives:

$$\mathbf{U}_5 = \begin{bmatrix} U_5 \\ 0 \end{bmatrix} = \begin{bmatrix} R_0 & P_0 \end{bmatrix} \begin{bmatrix} a \\ b \end{bmatrix} = G a_0,$$  \hfill (23)

where:

$$a_0 = \{a_1, a_2, \ldots, a_n, b_1, b_2, \ldots, b_m\},$$  \hfill (24)

$$\mathbf{U}_5 = \{u_1, u_2, \ldots, u_n, 0, 0, \ldots, 0\}.$$  \hfill (25)

From Eq. (23), we have:

$$a_0 = \left[ \begin{array}{c} a \\ b \end{array} \right] = G^{-1} \mathbf{U}_5,$$  \hfill (26)

A combination of Eqs. (12) and (26) gives:

$$u(X) = \begin{bmatrix} R^T(X) \\ P^T(X) \end{bmatrix} G^{-1} \mathbf{U}_5 = \mathbf{\Phi}^T(X) \mathbf{U}_5,$$  \hfill (27)

where:

$$\mathbf{\Phi}^T = \begin{bmatrix} \psi_1(X) \\ \psi_2(X) \\ \vdots \\ \psi_n(X) \\ \psi_{n+1}(X) \\ \vdots \\ \psi_{n+m}(X) \end{bmatrix}.$$  \hfill (28)

and the vector of shape functions is as follows:

$$\mathbf{\Phi} = \begin{bmatrix} \psi_1(X) \\ \psi_2(X) \\ \vdots \\ \psi_n(X) \end{bmatrix}.$$  \hfill (29)

3. Mesh-free formulation for beam on elastic foundation

Using the beam theory, the governing differential equation for the centroidal line of the deformed beam resting on a two-parameter elastic foundation can be written as [3]:

$$EI \frac{\partial^4 y}{\partial x^4} + K_1 y - K_2 \frac{\partial^2 y}{\partial x^2} = q(x),$$  \hfill (30)

where $E$ is the modulus of elasticity for the constitutive material of the beam, $I$ is the moment of inertia for the cross section of the beam, $y$ is the deflection of the beam at any point, $x$, $q(x)$ is the distributed load on the beam, and $K_1$ and $K_2$ are the first (Winkler) and the second foundation parameters, respectively.

Eq. (30) can be solved directly by strong form based methods, such as the finite difference method [19], or by weak form based methods, such as the finite element method [20]. In this paper, a weak form based method, which is also a mesh-free procedure, is implemented to solve the equation. In the present approach, the beam is considered as a two dimensional medium in a plane strain condition, and it is simulated separately from the elastic foundation. Considering the problem domain entirely (i.e. beam and elastic foundation), the total potential energy functional can be expressed as:

$$\Pi = \Pi_b + \Pi_f + \Pi_e,$$  \hfill (31)

where $\Pi_b$ and $\Pi_f$ are the elastic strain energy of the beam and the foundation, respectively, and $\Pi_e$ is the potential energy related to the external forces. These functionals are [21]:

$$\Pi_b = \int_{\Omega_b} \frac{1}{2} \varepsilon_b^T \sigma_b \, d\Omega_b,$$  \hfill (32)

$$\Pi_f = \int_{\Omega_f} \frac{1}{2} \varepsilon_f^T \sigma_f \, d\Omega_f,$$  \hfill (33)

$$\Pi_e = - \int_{\Gamma_b} U^T \mathbf{T} \, d\Gamma_b - \int_{\Omega_b} U^T \mathbf{b} \, d\Omega_b,$$  \hfill (34)

where $\varepsilon_b$ and $\sigma_b$ are, respectively, the strain and stress tensors related to the beam,

$$\varepsilon_b = \{\varepsilon_x, \varepsilon_y, \gamma_{xy}\}, \quad \sigma_b = \{\sigma_x, \sigma_y, \tau_{xy}\}.$$  \hfill (35)

$\varepsilon_f$ and $\sigma_f$ are, respectively, the strain and stress tensors across the foundation, $U$ is the displacement vector, $\mathbf{T}$ is the prescribed boundary traction and $\mathbf{b}$ is the body force vector. $\Omega_b$ and $\Omega_f$ stand for the beam and foundation domains, respectively.
is the beam boundary, on which the external tractions are imposed. It should be noted that subscripts $B$ and $F$ stand for the beam and foundation media, respectively.

Considering Hooke’s law (i.e. $\sigma = D \varepsilon$), the variational form of Eq. (31) can be written as:

$$\delta I = \iint_{\Omega_B} (\delta \varepsilon_B)^T D_B \varepsilon_B \; d\Omega_B + \iint_{\Omega_F} (\delta \varepsilon_F)^T D_F \varepsilon_F \; d\Omega_F$$

$$\quad - \int_{\Gamma} (\delta U)^T b \; d\Gamma - \int_{\Gamma} (\delta U)^T T \; d\Gamma = 0,$$

(36)

where $D_B$ and $D_F$ are the elasticity matrices for the beam and foundation, respectively. The elasticity matrix for beam material in a plane strain condition can be written as [21]:

$$D_B = \frac{E(1-\nu)}{(1+\nu)(1-2\nu)} \begin{bmatrix} 1 & \frac{\nu}{1-\nu} & 0 \\ \frac{\nu}{1-\nu} & 1 & 0 \\ 0 & 0 & \frac{1-2\nu}{2(1-\nu)} \end{bmatrix}, \quad (37)$$

where $E$ and $\nu$ are elasticity modulus and Poisson ratio, respectively. The elasticity matrix for the foundation will be derived later in this section.

The discrete form of equations can be obtained by imposition of the strain-displacement relation and Eq. (8) into Eq. (36). For the beam material, the strain-displacement relation can be easily written as:

$$\varepsilon = B U_B,$$

(38)

where:

$$B = \begin{bmatrix} B_1 & \tilde{B}_2 & \cdots & \tilde{B}_i & \cdots & \tilde{B}_m \end{bmatrix}, \quad (39)$$

and:

$$\tilde{B}_i = \begin{bmatrix} \frac{\partial \psi_i}{\partial x} & 0 & \frac{\partial \psi_i}{\partial y} \\ 0 & \frac{\partial \psi_i}{\partial x} & \frac{\partial \psi_i}{\partial y} \end{bmatrix} \begin{bmatrix} \frac{\partial \psi_i}{\partial x} & \frac{\partial \psi_i}{\partial y} \\ \frac{\partial \psi_i}{\partial x} & \frac{\partial \psi_i}{\partial y} \end{bmatrix}^T,$$

(40)

where $\psi_{ik} = \psi_k(x_i)$ is the shape function of the $k$th node at the $i$th node in the support domain. However, the derivation of the stress–strain relation and also, the strain-displacement relation in the foundation layer, requires some manipulation.

As shown in Figure 2, the two-parameter elastic foundation can be considered as a layer with two stiffness coefficients along two orthogonal directions (i.e. $K_s$ and $K_n$). According to Figure 2, the relative deformation vector, $\delta$, at point $P$ between the top and bottom surfaces can be related to the displacements of points $A$ and $B$. Hence, it can be written as:

$$\delta = \begin{bmatrix} \delta_s \\ \delta_n \end{bmatrix} = \begin{bmatrix} u_A - u_B \\ u_A - u_B \end{bmatrix}, \quad (41)$$

where $\delta_s$ and $\delta_n$ are, respectively, the shear and normal relative displacement at point $P$. $\bar{U}_A$ and $\bar{U}_B$ are the displacement vectors in the local coordinate $n-s$ at points $A$ and $B$, respectively. However, as the location of point $B$ is fixed, the displacement components of $\bar{U}_B$ (i.e. $u_B$ and $v_B$) are both zero. Then, Eq. (41) can be re-written as:

$$\delta = \begin{bmatrix} \delta_s \\ \delta_n \end{bmatrix} = \begin{bmatrix} u_A - u_B \\ u_A - u_B \end{bmatrix}, \quad (42)$$

where $u_A$ and $v_A$ are, respectively, the displacement of point $A$ in the $s$ and $n$ directions.

Considering the relation between local and global coordinates, we have:

$$\bar{U}_A = L U_A,$$

(43)

where $L$ is coordinate transform matrix and $U_A$ is the displacement vector of point $A$ in the global coordinate.

As shown in Figure 2, the field variable (i.e. displacement) at point $A$ can be estimated by its nodal values at the nodes located in the compact support domain of point $A$. According to Eqs. (8), (42) and (43), the displacement of point $A$ can be written as:

$$\delta = B_f U_A,$$

(44)

where $U_A$ is the displacement vector, composed of displacement at all nodes in the compact support domain of point $A$, and:

$$B_f = L \Phi.$$

(45)

Neglecting the normal strain component in the $s$ direction, the strain vector in the local coordinate can be defined as:

$$\varepsilon = \frac{1}{h} \delta,$$

(46)

where $h$ is the virtual thickness assumed for the foundation layer and $\varepsilon = \begin{bmatrix} \gamma_{ns} \end{bmatrix}$ in which $\gamma_{ns}$ is the normal strain in the direction $n$, and $\gamma_{ns}$ is the shear strain. By substituting Eq. (44) into Eq. (46), the relation between the strain and nodal displacement can be obtained as:

$$\varepsilon = B_f U_A,$$

(47)

where:

$$B_f = \begin{bmatrix} \frac{1}{h} \end{bmatrix} B_f.$$

(48)

To evaluate the stiffness matrix related to the elastic foundation, the relation between stress and strain in this region is also needed. According to Figure 2, the relation between the stress vector and relative deformation can be written as:

$$\sigma = D_f \delta,$$

(49)

where the stress vector, $\sigma$, consists of the normal stress, $\sigma_n$, and the tangential stress, $\tau$, in the foundation region.

$$\sigma = \begin{bmatrix} \tau & \sigma_n \end{bmatrix}^T.$$

(50)

Matrix $D_f$ can also be defined as:

$$D_f = \begin{bmatrix} K_s & 0 \\ 0 & K_n \end{bmatrix}.$$

(51)

Substituting Eq. (46) into Eq. (49) gives:

$$\sigma = D_f \delta.$$

(52)
where:
\[ D_f = h D_f. \] (53)

At this stage, all relations between stress, strain and displacement at the beam and foundation media are known, and Eq. (36) can be rewritten as:
\[
\begin{align*}
\int_{\Omega_B} \mathbf{B}_B^T \mathbf{D}_B \mathbf{B}_B \mathbf{d} \Omega_B &+ \int_{\Gamma_B} \mathbf{B}_B^T \mathbf{D}_f \mathbf{B}_f \mathbf{d} \Gamma_B \mathbf{U}_S = \int_{\Omega_B} \mathbf{B}_B^T \mathbf{b} \mathbf{d} \Omega_B + \int_{\Gamma_B} \mathbf{B}_B^T \mathbf{b} \mathbf{d} \Gamma_B.
\end{align*}
\] (54)

Assuming constant virtual thickness for the foundation layer, Eq. (54) can be written as:
\[
\left[ K_B + K_f \right] \mathbf{U}_S = \mathbf{F},
\] (55)

where:
\[
K_B = \int_{\Omega_B} \mathbf{B}_B^T \mathbf{D}_B \mathbf{B}_B \mathbf{d} \Omega_B,
\] (56)
\[
K_f = \int_{\beta} \mathbf{B}_f^T \mathbf{D}_f \mathbf{B}_f \mathbf{d} \beta,
\] (57)
\[
\mathbf{F} = \int_{\Omega_B} \mathbf{B}_B^T \mathbf{b} \mathbf{d} \Omega_B + \int_{\Gamma_B} \mathbf{B}_B^T \mathbf{b} \mathbf{d} \Gamma_B,
\] (58)

where \( \beta \) is the length parameter along the foundation layer.

4. Numerical study

4.1. Example 1. Beam on Winkler foundation

In this example, the well-known problem of a beam on a Winkler foundation is analyzed by the proposed mesh-free method. The problem has a closed form solution, which can be found in [9] and, hence, it is used as a benchmark for evaluation of the results of the proposed mesh-free method. As shown in Figure 3, a beam with 14 m length is considered on a Winkler foundation with the reaction modulus of 10 MN/m³. A 100 kN concentrated force is exerted at 2 m distance from the left hand side of the beam. The flexural rigidity of the beam is assumed 2604.167 kN·m², which is equivalent to a beam with 0.25 m thickness and a Young’s modulus of \( 2 \times 10^3 \) MPa at the plane strain condition. The Poisson ratio of the beam material is assumed 0.25 for numerical analysis.

To compare the results of numerical analysis with the exact solution results, a mesh-free model is constructed. As shown in Figure 4, the model consists of 58 nodes located along two parallel lines and a background mesh with 14 blocks for numerical integration. The results of analysis for both methods are depicted in Figure 5. It should be noted that the exact solution renders the deflection along the center line of the beam. Hence, the averaged values of the deflection (i.e. averaged values of deflection for the nodes located above and beneath the specified point along the center line of the beam) obtained from numerical analysis are used for comparison. As is obvious from Figure 5, there is excellent agreement between the results of the exact solution and the proposed mesh-free method, which confirms the accuracy and efficiency of the proposed numerical solution.

4.2. Example 2. Two parameter elastic foundation

A typical problem of a beam on a two-parameter elastic foundation is investigated in the present example. As shown in Figure 6, a beam with 10 m length and 1 m thickness is assumed. The elastic modulus and Poisson ratio of the beam material are 2 GPa and 0.25, respectively. The normal and shear stiffness modulus are, respectively, 15 and 10 MN/m³. The plane strain condition is assumed. Two 500 kN concentrated loads are exerted in a symmetric manner at 2 m distance from each end.

The problem is solved by the finite element program SIGMAW [22], as well as the proposed mesh-free method. The finite element modelling consists of two models: 10 elements and 40 elements (Figure 7). It is obvious that by increasing the number of elements, the accuracy of results will also increase. Hence, this can be a measure to investigate the accuracy of
the proposed mesh-free method. As depicted in Figure 8, the mesh-free model is constructed by the same number of nodes as the 10 elements finite element model. The background mesh consists of 10 blocks with 4 Gauss points in each block (Figure 9). The results of analyses are shown for the upper and lower surfaces of the beam in Figures 10 and 11. Due to the symmetry of the model, only results for one half of the beam are demonstrated. As is obvious from the figures, the proposed mesh-free method offers acceptable results that are even more accurate than the results of finite element analysis with the same order of nodes. To verify this claim, quantitatively, the Mean Root Square Errors (MRSE) between different solutions are compared with each other in Table 1. In this table, the second column consists of the MRSE between the FEM with 10 elements (FEM1) and the FEM with 40 elements (FEM2), for the horizontal and vertical displacement of nodes. The third column of Table 1 shows the MRSE between MFM and FEM2 for the same components of displacement in the previous column. As obvious from values shown in Table 1, results of the mesh-free method are closer to the results of the exact solution, remembering the fact that by increasing the number of elements in FEM, the results get closer to the exact solution.

5. Conclusion

A mesh-free method is implemented for the analysis of a beam on a two parameter elastic foundation in the hope of opening a new outlook in the application of such approaches to structural analysis. The beam and elastic foundation are simulated separately and no element is used in each part. The beam is simulated by two or three sets of nodes aligned along parallel lines. There is no connectivity between nodes, and they can be added or omitted easily. This feature gives much credit to mesh-free methods, especially in adaptivity analysis. The elastic foundation is simulated by the linkage element concept, and there is no need for nodes or elements in the traditional sense. Hence, the stresses across the foundation layer can be determined readily without any additional post processing manipulation. Besides all these benefits, the accuracy of results is also acceptable, which is even better than finite element results with the same order of nodes.
Figure 11: (a) Horizontal and (b) vertical displacement of the nodes located at the upper edge of the beam.

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