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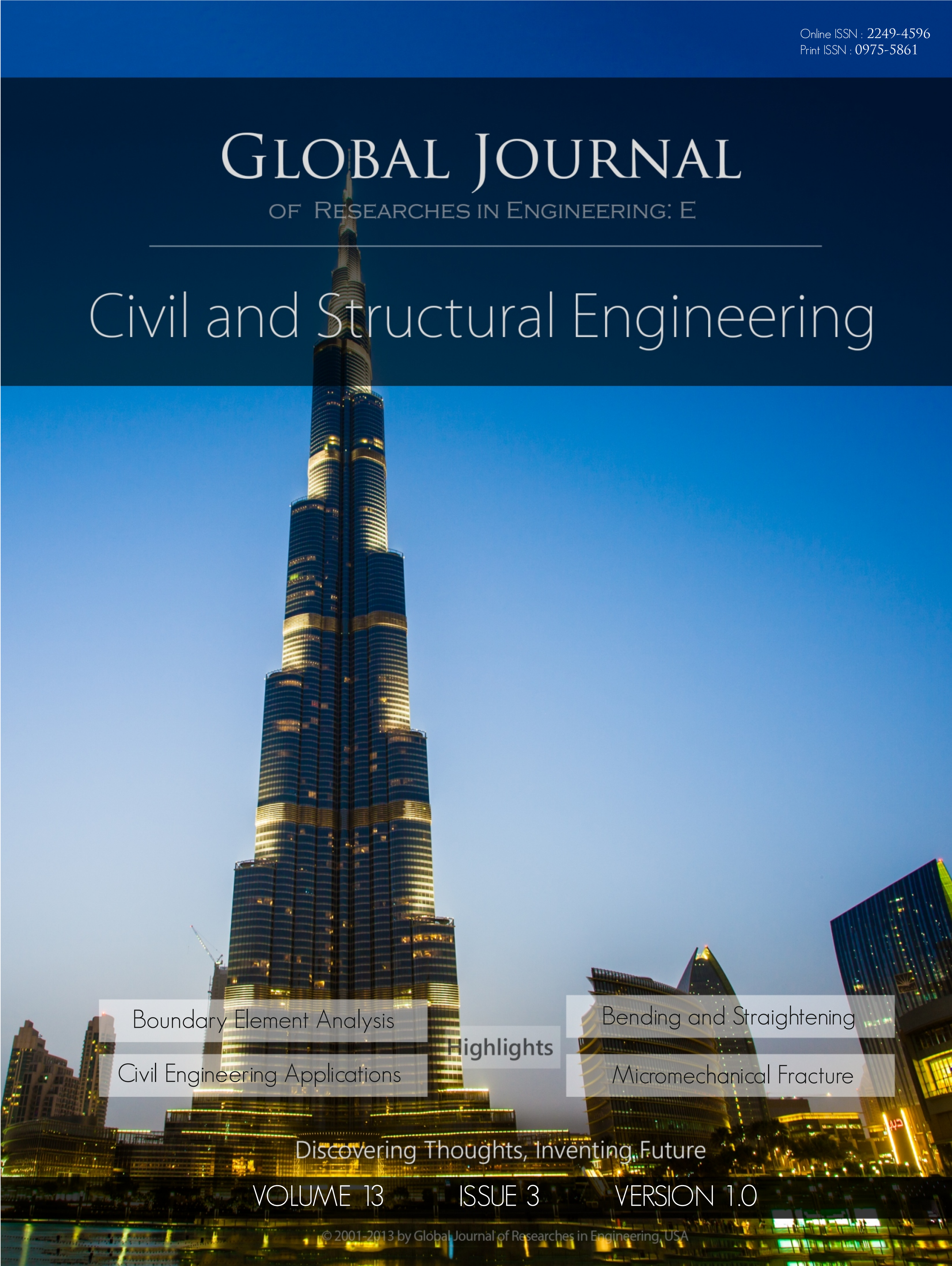

Highlights

- Boundary Element Analysis
- Civil Engineering Applications
- Bending and Straightening
- Micromechanical Fracture

Discovering Thoughts, Inventing Future

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Boundary Element Analysis of Pile Groups in Sand

By Mohammed Y. Fattah, Kais T. Shlash & Madhat S. Al-Soud

University of Technology, Baghdad, Iraq

Abstract - This paper is devoted to make use of the boundary element method as a practical problem solving tool to analyze the soilpile interaction problems. A computer program was adopted for the analysis process. It is a computer package called PGroupN which is concerned mainly with the analysis of the pile group problems. A non-linear soil model has been adopted with this program to assess the pile-soil interaction within the group. Some parametric studies were carried out with this problem including the pile diameter, pile length, spacing to diameter ratio, soil type (sand) and the thickness of stratum.

It was found that for 9 pile group analyzed, the corner pile carries about 4% more than the average load, while the border and the center piles carry 2% and 10% less than the average load, respectively. As the ratio of pile spacing to diameter (S/D) increases, the difference between loads on each pile decreases, especially between the corner and the center piles.

Keywords : boundary element, pile group, sand, pile load.

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BOUNDARY ELEMENT ANALYSIS OF PILE GROUPS IN SAND

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Boundary Element Analysis of Pile Groups in Sand

Mohammed Y. Fattah [α], Kais T. Shlash [σ] & Madhat S. Al-Soud [ρ]

Abstract - This paper is devoted to make use of the boundary element method as a practical problem solving tool to analyze the soil- pile interaction problems. A computer program was adopted for the analysis process. It is a computer package called PGroupN which is concerned mainly with the analysis of the pile group problems. A non-linear soil model has been adopted with this program to assess the pile-soil interaction within the group. Some parametric studies were carried out with this problem including the pile diameter, pile length, spacing to diameter ratio, soil type (sand) and the thickness of stratum.

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Keywords : boundary element, pile group, sand, pile load.

I. Boundary Element Technique

While the finite element calculations constitute the major part of the computer studies and considered the most popular technique, the boundary element method (BEM) has been well examined as complementary or alternative system of analysis, with a view of reduce program running costs and enabling larger problems to be tackled.

The BEM is a boundary integral equation technique. The problem boundaries are discretized so that a singular solution of the governing differential equation can be integrated around them, yielding an appropriate source distribution which will generate the specified boundary conditions. Alternatively, the method may be regarded as a 'super' finite element technique, where each element models a homogenous zone (Hobbs et al., 1978).

Early attempts at employing numerical solution procedures for the solution of boundary integral equations have developed into two distinct and parallel ways. One of these is an intuitive, physical approach and the other is a more mathematical treatment based on concepts of classical potential theory (Banerjee and Butterfield, 1981) (Crouch and Starfield, 1983) (Venturini, 1983), (Beer, 1986) and (Beer and Watson, 1992).

a) *Boundary Element Method in pile-soil Interaction Problems*

Poulos (1971a) proposed integral equations for an elastic solution of laterally loaded pile. This problem is based on Mindlin's solution (Mindlin, 1936) for a point load in a homogeneous, isotropic elastic half space. The pile response shown in Figure 1 was calculated by integrating Mindlin's equation over the corresponding area of the soil. In Figure 1a, the pile is discretized into a number of elements. The transition of load through the pile is shown in Figure 1b, while the pile deformation (w) is shown in Figure 1c. The method could be extended to study the behavior of pile groups (Poulos, 1971b).

The integral equation mentioned above, was extended by Butterfield and Banerjee (1971b) using a BEM to treat a soil continuum and pile as two separate domains whose boundaries were discretized into a finite number of elements. A set of fictitious tractions were assumed at the pile – soil interface. It had been shown by Butterfield and Banerjee (1971b) that these fictitious tractions were identical to the real ones for pile slenderness ratio greater than five. Each boundary element was associated with known tractions and displacements. Some of these were known over parts of the boundary, the rest of them were computed using Kelvin' solutions (Lancellotta, 2009). Once these boundary values were obtained, the displacements and tractions at any point inside the domain could be computed.

Butterfield and Banerjee (1971a) presented an elastic analysis for the general compressible pile group problem including a rigid smooth ground contacting cap. The problems were formulated as an integral equation developed from Mindlin's analysis (Mindlin, 1936) for a point load embedded within a semi-infinite ideal elastic half space. By distributing such point loads over the pile cap – supporting medium interface and the pile shaft and pile base – medium interface, an integral representation was obtained given the vertical displacement at all points in the medium in terms of fictitious stress intensities.

Chin (2004) derived elastic design charts for axial pile settlement response from the elastic response simplified BEM for piles imbedded in a two-layer soil

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continuum. These charts cover the practical ranges of pile and soil parameters as shown in Figure 2:

b) Application of the Boundary Element Method to Soil-Pile Interaction Problems

This paper presents the results of a numerical analysis for the solution of soil-pile interaction problems using the BEM.

A problem was chosen to investigate the behavior during the variation of different parameters. A summary of the analysis procedure is presented here followed by a comprehensive discussion to all the obtained results in order to evaluate the work, as much as possible, according to the adopted researches.

Because of the particulate nature of the mineral skeleton of the soil, the stress-strain behavior of the soil is exceedingly complex. One way out from this complexity is to use concepts and formulas from the theory of elasticity. This means that the actual non-linear behavior of soil is linearized which may lead to a conservative design. Thus, the non-linearity represents the realistic behavior of the soil and it is adequate to simulate its general problems.

Many researchers emphasized the importance of considering soil non-linearity in routine design. For pile group problems, this issue has not been satisfactorily addressed yet, and current design practice is still generally based on linear approaches (Basile, 2003). The main drawback to the application of linear models to pile group problems is that they ignore the non-linear load-deformation characteristics of soil and hence misrepresent the force in piles, specifically by giving higher stresses in group corners. The cost of this in practice is high and there is an urgent need in industry for efficient non-linear analysis method (Basile, 2003).

A reasonable compromise between excessive complexity and unacceptable simplicity is provided by the BEM, in which the pile-soil interface is discretized and the characteristics of soil response are represented in a lumped form by ascribing the behavioral features of the soil to the interface elements (Poulos, 1989).

II. Method of Analysis

The analysis with the program PGroupN is based on complete non-linear BEM formulation. The analysis involves discretization of only the pile-soil interface into a number of cylindrical elements, while the base is represented by a circular (disc) element. The method employs a subtracting technique in which the piles and the surrounding soil are considered separately and then compatibility and equilibrium conditions are imposed at the interface.

The external load is applied incrementally and, at each increment, a check is made that the stress state at the pile-soil interfaces does not violate the yield criteria. This is achieved by specifying the limiting stresses of the soil for the axial pile shaft capacity, and end-bearing resistance. The elements of the pile-soil interface which have yielded can take no additional load and any increase in load is therefore redistributed between the remaining elements until all elements have failed. Thus, by successive application of loading increments, the entire load-displacement relationship for the pile group is determined.

a) BEM Program PGroupN

Repute calculation engine is the leading-edge program PGroupN which provides a complete 3D non-linear boundary element solution of the soil continuum. This overcomes limitations of traditional interaction-factor methods and gives more realistic predictions of deformations and the load distribution between piles (Basile, 2003).

This program is based on a complete boundary element formulation extending an idea first proposed by Butterfield and Banerjee (1971, a). The method employs a substructuring technique in which the piles and the surrounding soil are considered separately and then compatibility and equilibrium conditions are imposed at the interface. Given unit boundary conditions, i.e. pile group loads and moments; these equations are solved, thereby leading to the distribution of stresses, loads and moments in the piles for any loading condition (Geocentrix Ltd, 2002).

b) Choice of Soil Parameters

The choice of soil parameters for PGroupN is simple and direct: for a linear analysis, it is only necessary to define two soil parameters whose physical interpretation is clear, i.e. the soil modulus (E) and Poisson's ratio (ν). If the effects of soil non-linearity are considered in elastic-plastic analysis, the strength properties of soil need also to be specified, i.e. the undrained shear strength (c_u) for cohesive soils and the angle of friction (ϕ) for cohesionless soils. Thus the applied method, by taking into account the continuous nature of pile-soil interaction, removes the uncertainty of *t-z* and *p-y* approaches and provides a simple design tool based on conventional soil parameters (Basile, 2003).

c) Nonlinear Soil Model

Soil is not a linear material. The relations between stress and strain are much more complicated than the simple, linear elastic material. Therefore, in order to represent geotechnical problems realistically, some form of nonlinear relation must be used (Christian and Desai, 1977).

The widely used function for simulation of stress-strain curves in the finite element analysis was formulated by Chang and Duncan (1970), and Duncan and Chang (1970) using Kondner's (1963) finding that the plot of stress versus strain in a triaxial compression

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test is very nearly a hyperbola. Figure 3 illustrates such relation, which can be stated in equation form:

$$\sigma = \frac{\varepsilon}{b + a\varepsilon} \tag{1}$$

or

$$\frac{\varepsilon}{\sigma} = b + a\varepsilon \tag{2}$$

In these equations, the subscripts have been removed from the stresses and strains for clarity, so that σ and ε represent vertical stress and strain, respectively. The latter form of the equation plots as a straight line (Figure 4) and, conversely, a plot with axis ε/σ and ε can be used to check whether the data from a test do fit a hyperbola or to find the parameters of the hyperbola from the test data.

To find the tangent modulus, it is first helpful to observe that at very small strains:

$$\sigma = \frac{\varepsilon}{b} \tag{3}$$

So that 1/b is the initial Young's modulus E. At large strains, the relation becomes:

$$\sigma = \frac{1}{a} \tag{4}$$

So that 1/a is the compressive strength; actually, 1/a is the asymptote. Since the compressive strength will be reached before the curve becomes asymptotic as shown in Figure 2, it is customary to require the compressive strength *s* to be R_f /a, where R_f is the failure ratio. Thus

$$a = \frac{R_f}{s} \tag{5}$$

Equation (1) can also be solved for ε:

$$\varepsilon = \frac{b\sigma}{1 - a\sigma} \tag{6}$$

The tangent modulus at any level of stress or strain is:

$$E_t = \frac{\partial\sigma}{\partial\varepsilon} = \frac{b}{(b + a\varepsilon)^2} = \frac{1}{b}(1 - a\sigma)^2 = E_i(1 - \frac{R_f\sigma}{s})^2 \tag{7}$$

For a Mohr-Coulomb material at failure:

$$(\sigma_1 - \sigma_3)_f = \frac{2\sigma_3 \sin\phi + 2c\cos\phi}{1 - \sin\phi} \tag{8}$$

The term σ/s is the ratio between the existing σ_1-σ_3 and *s* that would be available for the existing σ_3. The ratio is:

$$\frac{\sigma}{s} = \frac{(\sigma_1 - \sigma_3)(1 - \sin\phi)}{2\sigma_3 \sin\phi + 2c\cos\phi} \tag{9}$$

The tangent modulus now becomes:

$$E_t = E_i\left[1 - \frac{R_f(\sigma_1 - \sigma_3)(1 - \sin\phi)}{2\sigma_3 \sin\phi + 2c\cos\phi}\right]^2 \tag{10}$$

The initial modulus has been found to vary with the confining pressure:

$$E_i = Kp_a\left(\frac{\sigma_3}{p_a}\right)^n \tag{11}$$

where p_a is the atmospheric pressure,

K and *n* are constants to be determined.

The complete relation then becomes:

$$E_t = Kp_a\left(\frac{\sigma_3}{p_a}\right)^n\left[1 - \frac{R_f(1 - \sin\phi)(\sigma_1 - \sigma_3)}{2c\cos\phi + 2\sigma_3 \sin\phi}\right]^2 \tag{12}$$

The PGroupN analysis adopts a non-linear model, which follows the well–established hyperbolic relationship between stress and strain proposed by Duncan and Chang (1970) and also applied to pile problems by Poulos (1989). This simple relationship assumes that the soil Young's modulus (E_t) varies with the stress level at the pile-soil interface, i.e. it is a function of the initial tangent soil modulus (E_i), the hyperbolic curve-fitting constant (R_f), the current pile-soil stress (*t*) and the limiting value of pile-soil stress (τ_{lim}), as shown in Figure 5. The hyperbolic curve fitting constant R_f defines the degree of non-linearity of the stress-strain response and can range between zero (an elastic-perfectly plastic response) and one (an asymptotic hyperbolic response in which the limiting pile-soil stress is never reached. The best way to determine the value of R_f is by fitting the PGroupN load-deformation curve with the data from the full-scale pile load test. In the absence of any test data, the value of R_f can be initially estimated based on experience.

d) Soil Domain

The BEM involves the integration of an appropriate elementary singular solution for the soil medium over the surface of the problem domain, i.e. the pile-soil interface. With reference to the present problem, the well-established solution of Mindlin (1936) for a point load within a homogeneous, isotropic elastic half space has been adopted, yielding:

$$\{u_s\} = [G_s]\{t_s\} \tag{13}$$

Where $\{u_s\}$ are the soil displacements, $[G_s]$ is the flexibility matrix obtained form Mindlin's solution and

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$\{t_s\}$ are the soil tractions. The singular part of the $[G_s]$ matrix is calculated via analytical integration of the Mindlin functions.

e) Pile Domain

If the piles are assumed to act as simple beam-columns which are fixed at their heads to the pile cap, the displacements and tractions over each element can be related to each other via the elementary beam theory, yielding:

$$\{u_p\} = [G_p]\{t_p\} + \{B\} \qquad (14)$$

Where $\{u_p\}$ are the pile displacements, $\{t_p\}$ are the pile tractions, $\{B\}$ are the pile displacements due to unit boundary displacements and rotations of the pile cap, and $[G_p]$ is the matrix of coefficients.

f) Solution of the System

The above soil and pile equations are coupled via compatibility and equilibrium constraints at the pile-soil interface. Thus, by specifying unit boundary conditions, i.e., unit values of vertical displacement, horizontal displacement and rotation of pile cap, these equations are solved, thereby leading to the distribution of stresses, loads and moments in the piles for any loading condition.

g) Extension to Non-linear Soil Behavior

Non-linear soil behavior is incorporated, in an approximated manner, by assuming that the soil Young's modulus varies with the stress level at the pile-soil interface (Basile, 2003). A simple and popular assumption is to adopt a hyperbolic stress-strain relationship, in which case the tangent Young modulus of the soil E_t may be written as (Duncan and Chang, 1970; Poulos, 1989):

$$E_t = E_i \left(1 - \frac{R_f\, t}{t_{\lim}}\right)^2 \qquad (15)$$

Where E_i is the initial tangent soil modulus,
R_f is the hyperbolic curve-fitting constant,
t is the pile-soil shear stress, and
t_{lim} is the limiting value of the pile-soil stress.

Thus, the boundary element equations described above for the linear response are solved incrementally using the modified values of soil Young's modulus of Equation (15) and enforcing the conditions of yield, equilibrium and compatibility at the pile-soil interface.

Different values of R_f should be used for the axial response of the shaft and base, and for the lateral response of the shaft. For the axial response of the shaft, there is relatively small amount of nonlinearity, and values of R_f in the range 0-0.5 are appropriate, (Polous, 1989). The axial response of the base is highly nonlinear, and the value of R_f in the range 0.9-0.99 is recommended (Poulos, 1989).

h) General Description of the Problem

Group behavior is very complex. The response of each pile is modified by the stress condition imposed on the soil by other members of the group. Therefore, the behavior is generally dependent on the pile spacing and length, relative stiffness of the piles, number of piles in the group, in addition to the soil conditions.

In order to carry out parametric study and investigate the influence of these parameters on the behavior of the piles, it is essential to start with basic problems. The problem which is chosen to be studied is a system of pile group under axial load.

Two sets of pile groups consisting of 4 and 9 piles with circular cross sections are embedded in the soil. These pile groups were assigned different internal and external variables in order to study the behavior of the piles. The internal variables refer to pile diameter, pile length and spacing between piles, while the external variables refer to the applied load, soil types and their thickness.

It is assumed that the pile cap is fully rigid and not in contact with soil. The free-standing length, which represents the distance from the ground surface to the bottom of the cap, is assigned to be 0.5 m. This means that embedded length is reduced by 0.5 m and the interface elements are not considered within this gap. Unlike the 4 pile group, the behavior of a single pile in a group of 9 piles is associated with its location within the group. The key of identification of the piles in that group is shown in Figure 6.

The soil is assumed to be homogeneous and its parameters are based on subsoil idealization. The soil layer rests on a rigid base. The level of water table is 1.0 m below the ground surface. The piles are not based on the rigid layer which classified them as floating piles. A simple idealization of the pile-soil system is shown in Figure 7.

i) Parametric Study

Any designer is normally interested in the following aspects of the behavior of the pile groups:

- Evaluation of the collapse load;
- Calculation of the settlement which leads to select a suitable factor of safety in the design; and
- The distribution of stress along the piles so that it can provide adequate reinforcement in the piles.

The above targets with their simple statements represent a summary for the long analysis journey through many parameters which affect the pile group behavior. Some of these parameters adopted in this study are those incorporated with the nonlinear analysis based on the BEM. Table 1 shows an outline of the analysis program for the pile group problem.

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j) Pile Length

The length of the pile (L) plays an important role in increasing the bearing capacity of the group. The analysis begins with a pile length of 10 m and reaches a length of 25 m with increments of 5 m.

k) Pile Diameter

It is well known that the pile cross sectional area affects the capability of pile to sustain the loads. This parameter is taken into account during the analysis of the pile with its circular cross section. Six diameters are chosen to be the cases under study represented by (0.5 m, 0.6 m, 0.7 m, 0.8 m, 0.9 m and 1.0 m), respectively.

l) Spacing between Piles

Due to pile-soil-pile interaction, the group of piles tends to settle more than a proportionally loaded single pile. This is because neighboring piles are within each others, so each pile interacts with the surrounding piles which transfer the stresses to the other piles (Basile, 2003). Thus, the spacing between piles (S) is chosen as another parameter in this study. The spacing is usually correlated with the pile diameter, so the values of spacing are S=2D, S=3D, S=4D and S=5D.

m) Applied Axial Load

The failure of pile groups under axial loading has been extensively examined (Basile, 2003). The applied load is increased gradually until the stresses along the pile reach the limiting state.

n) Soil Type

The soil type is sand taken from Kerbala city in Iraq with the properties determined by Ghalib, (1975). The sand was washed between the No.30 (0.59 mm) and No.50 (0.297 mm) sieves to obtain a more uniform material which would not segregate during sample preparation. A special drained triaxial test on a cylindrical sand samples (76 mm height and 76 mm diameter) was used. The axial force was increased up to an axial strain of 20%. The maximum void ratio was obtained by slowly pouring a sample of the dry sand into a Proctor mould from very low height. Minimum void ratio was determined by fabricating the sand in the mould under (0.141 kg/cm^2) surcharge as specified by the ASTM standards. The triaxial chamber was first filled with de-aired water, and then the soil specimen was first consolidated to equilibrium under all around confining pressure. The confining pressures were 100, 150 and 250 kN/m^2. Volumetric changes were measured during the application of the cell pressure. Table 2 illustrates the properties of the soil to be considered in this study. The stress – strain curves from triaxial test are shown in Figure 8.

This sand was used in this simulation because it has good properties and detailed tests on this sand are well documented.

o) Soil Thickness

The groups of piles are embedded in a soil layer with a thickness (H) of 30 m, 45 m, 60 m and 75 m respectively, in order to study the effect of soil layer thickness. The program shows incapability to deal with cases where the pile is standing directly upon the bottom rigid layer (i.e. H=L).

III. Results and Discussion

The results were normalized by taking the factor K_p (stiffness factor) represented by the equation:

$$Kp = \frac{P}{GWD} \qquad (16)$$

Where P = axial load,

G = shear modulus of the soil medium = $\dfrac{E}{2(1+\nu)}$

W = displacement of head of pile, and

D = shaft diameter.

The non-dimensional stiffness factor (K_p) for 4 pile group embedded in sand is shown in Figures 9 to 11 as a function of the length to diameter ratio (L/D). This factor gives an indication to the load-settlement relationship during the loading process. Since the cap is assumed to be fully rigid, thus all the piles in the group settle by the same amount. The piles are arranged at different spacing for each load increment to find out their effects on the group behavior.

It is well known that increasing the pile length means that more shaft resistance is generated at the pile-soil interface along its length; moreover, the pile is penetrating the soil to settle down on a stiffer stratum. This increase in the pile capacity, if the load is still the same, leads to a reduction in the group settlement which is clearly shown in these figures. At an applied load of (8000 kN), Figure 9 checks that an increase in the pile length form (10 m) to (25 m) results in 53% decrease in settlement, whereas this percentage becomes 63% as the load increases up to 12000 kN. One can also notice the loss of the first points (i.e. at L=10 m) in some figures. This means that as the applied load is continuously increased, the shorter piles will be failed first.

These figures also show the role of the pile diameter during the loading process. The enlargement of the pile diameter produces a reduction in the intensity of loads carried by each pile in the group. The result is a reduction in loads to be transferred to the soil and less settlement is to be produced.

Figures 9 to 11 also show the influence of spacing between piles in a group. The behavior of a single pile in the group is associated with the others by means of strong bond, which is the cap. Thus, as the distance between the neighboring piles decreases, the interaction between these adjacent piles increases and vise versa. The results indicate that settlement of the short pile of a length (10 m) is decreased by 11% due to

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doubling the spacing from (S=2D) to (S=4D) while for pile of a length (25 m), the doubling process did not make a considerable effect.

Figure 12 shows the effects of the presence of a pile within a group as compared with a single pile having the same properties and dimensions. It is clearly shown that the interaction between piles in a group tends to increase the settlement more than the single pile even if they carry the same amount of loads.

Figure 13 shows the variation of the skin friction along a single pile in the group. Here, the pile length is chosen to be (25 m) and the friction at the pile interface is taken as the accumulative forces along the pile length. In close examination of this figure, one can notice the high convergence among the curves at each pile diameter. This gives an impression that the frictional resistance at the shaft is developed first until it reaches the limiting stress, then the pile base will take the rest of the applied load.

It is evident that the shaft resistance increased with increasing pile diameter due to the increase in the surface area which is in contact with the surrounding soil. This will contribute in a more frictional resistance to be added to the pile capacity with each diameter increment. The direct relation with depth reflects the influence of the effective overburden stress which increases with depth.

Figure 14 shows the corresponding plots of the percentage of load carried by the pile base. For a group of 4 piles and connected with a square cap, the load on each pile is equal since the total load is applied axially at the center of group. The miscellaneous trends of the curves from the linear to nonlinear and, return back finally to the linear path indicate the sharing process between the shaft and the base of the pile. The middle portion of the curves illustrates the beginning of the shaft resistance to take its share of the total load until it is fully mobilized.

This feature is compatible with the load-settlement curves shown in Figure 15. The nonlinear path is approximately diminished as the pile reaches its ultimate bearing capacity.

Figures 16 to 18 show the plot of the non-dimensional stiffness (K_p) as a function of pile depth. These relations are given for 9 pile group embedded in sand and subjected to sequence of axial loads applied at the center of the cap. Generally, the curves follow similar trends as in the case of 4 pile group during the variation of different parameters. Although the two mentioned series of the pile groups have similar trends but they are different in their values. The results are in good agreement with those of Butterfield and Banerjee (1971 b) and Paiva and Trondi (1999).

As a supplement to these figures, Figure 19 shows the relation of the pile head settlement against the total applied load for 9 pile group embedded in sand. A comparison with the load-settlement curves of 4 pile group shown in Figure 15 clarifies the influence of increasing the number of piles in the group in causing a larger amount of settlement. This result makes sense because as the number of piles in the group increases with constant spacing, the interaction between the adjacent piles certainly increases and the effects are more pronounced.

A rigid cap commonly offers uniform displacements for the group, but, on the other hand, a nonuniform distribution of loads appears. This fact is clearly shown in Figure 20 which indicates the load distribution on group of 9 piles embedded in sand. The load on each pile (P) is normalized by the average load of all piles in the group (P_{av}). It is evident that the greatest loads are carried out by the corner piles, followed by the border piles (mid-side piles) and the center pile which is compatible with Basile (2003) and Matos et al. (2005). The reason is that the center pile, due to pile-soil-pile interaction, will need a smaller amount of load to settle of the same amount as the corner pile. It can be seen that the corner pile carries about 4% more than the average load, while the border and the center piles carry 2% and 10% less than the average load, respectively. As the ratio of pile spacing to diameter (S/D) increases, the difference between loads on each pile decreases, especially between the corner and the center piles.

One can find a noticeable effect of the pile spacing to diameter ratio (S/D) on the load distributed for all the piles in the group but for the border pile. As the spacing ratio increases form (S/D=2) to (S/D=5), the difference between the corner and the center piles is reduced from 13% to 6% with approximately a horizontal path for the border piles. At a spacing ratio (S/D=3), the effect of length seems to be diminished through the high convergence between the different lengths behind this ratio. This refers to that moving the pile apart from the adjacent piles in a group tends to reduce the interaction among them which allow the pile to go through more independence.

Ideally, for an axially loaded pile group, all piles will carry the same amount of load as the total applied load approaches the ultimate load capacity of the group (Basile, 2003). This feature becomes clearer in Figure 17 as the total load increases towards its ultimate value.

Figure 21 illustrates the share of load carried by the base of each pile in the group. It can be seen that the pile's base takes different portions of loads according to their locations but follow the same trend during the gradual increase in the applied load. This may lead to think, as it was mentioned before in the problem of 4 pile group, that the shear stress along the pile's interface is developed equally regardless of the number and location of piles.

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IV. Conclusions

1. A rigid cap offers uniform displacements for all the piles in the group, but on the other hand, a non uniform distribution of loads appears. For the group of 9 piles, the maximum loads are carried by the corner piles, followed by the border piles and the center piles. This conclusion obtained by the boundary element analysis coincides with the same findings obtained by other approaches.
2. For 9 pile group analyzed, the corner pile carries about 4% more than the average load, while the border and the center piles carry 2% and 10% less than the average load, respectively. As the ratio of pile spacing to diameter (S/D) increases, the difference between loads on each pile decreases, especially between the corner and the center piles.
3. As the applied load reaches the ultimate capacity of the group, all the piles will share the same amount of load.
4. For a pile group embedded in sand, the shear stress along the pile interface increases gradually during the loading process until it become fully mobilized, then the pile base takes the rest of the applied load. This shear stress is found to be equally developed regardless of the number and the location of the piles in the group.

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Table 1 : Analysis program for pile group problem

Pile diameters are ranged from 0.5 m – 1.0 m with an increment of 0.1 m							
S=2D		S=3D		S=4D		S=5D	
L=10 m	H=30 m	L=10 m	H=30 m	L=10 m	H=30 m	L=10 m	H=30 m
	H=45 m		H=45 m		H=45 m		H=45 m
	H=60 m		H=60 m		H=60 m		H=60 m
	H=75 m		H=75 m		H=75 m		H=75 m
L=15 m	H=30 m	L=15 m	H=30 m	L=15 m	H=30 m	L=15 m	H=30 m
	H=45 m		H=45 m		H=45 m		H=45 m
	H=60 m		H=60 m		H=60 m		H=60 m
	H=75 m		H=75 m		H=75 m		H=75 m
L=20 m	H=30 m	L=20 m	H=30 m	L=20 m	H=30 m	L=20 m	H=30 m
	H=45 m		H=45 m		H=45 m		H=45 m
	H=60 m		H=60 m		H=60 m		H=60 m
	H=75 m		H=75 m		H=75 m		H=75 m
L=25 m	H=30 m	L=25 m	H=30 m	L=25 m	H=30 m	L=25 m	H=30 m
	H=45 m		H=45 m		H=45 m		H=45 m
	H=60 m		H=60 m		H=60 m		H=60 m
	H=75 m		H=75 m		H=75 m		H=75 m

Table 2 : Soil parameters of Kerbala sand for different models (after Ghalib, 1975)

a. Identification properties

Soil Properties	Value
Uniformity coefficient, Cu	1.41
Effective diameter, D_{10}	0.44 mm
Coefficient of curvature, Cc	0.36
Specific gravity, Gs	2.679
Min. void ratio, e_{min}	0.543
Max. void ratio, e_{max}	0.8043
Relative density, Dr %	80.835

b. Strength properties

Soil properties	Linear-Elastic model	Duncan and Chang model	Mohr- Coulomb model
E_i (kN/m^2)	124000 171500 220000	124000 171500 220000	124000 171500 220000
γ_{bulk}, (kN/m^3)	20.15	20.15	20.15
Poisson's ratio υ	0.32	0.32	0.32
σ_3 (kN/m^2)	100 150 250	100 150 250	100 150 250
Ø (degree)	-	40	40

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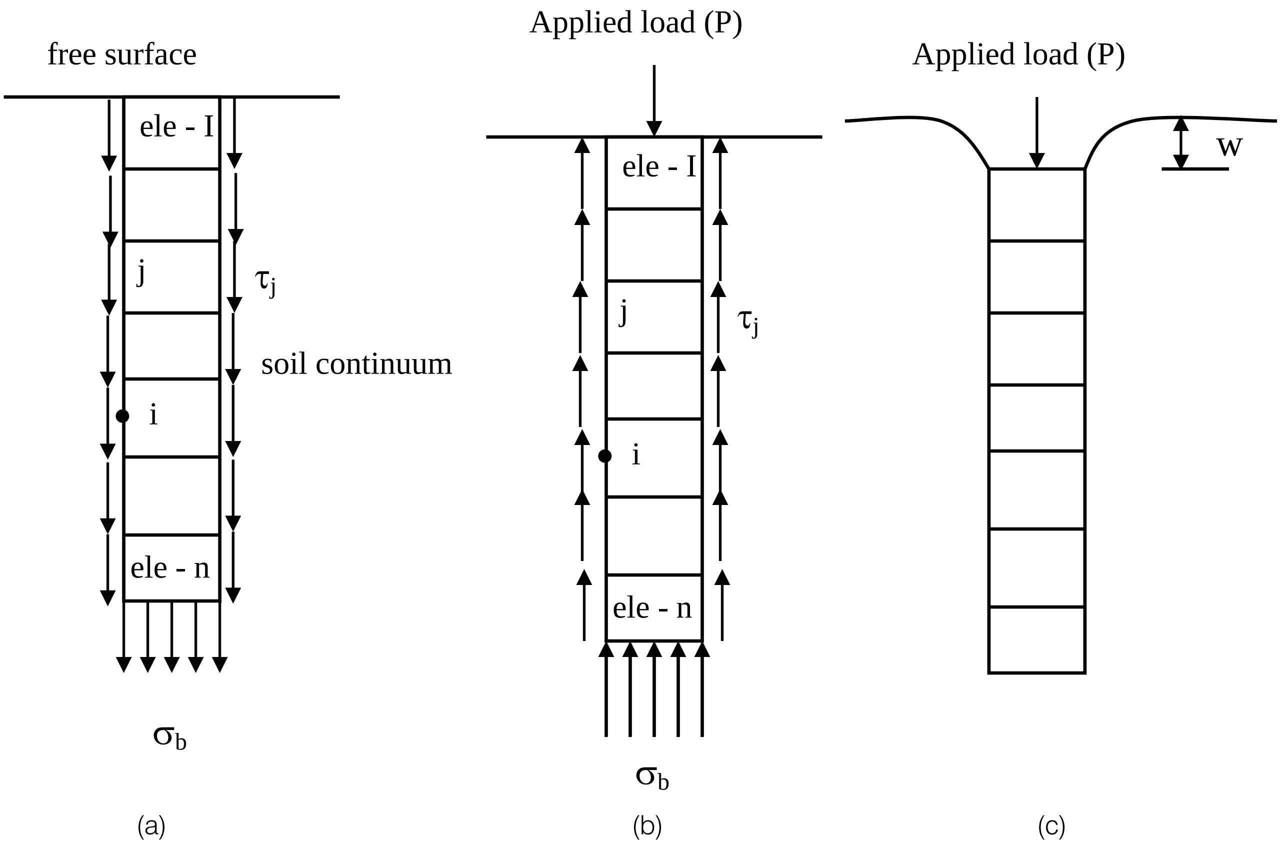

Figure 1 : Schematic diagram of the integral equation method (after Poulos, 1971a)

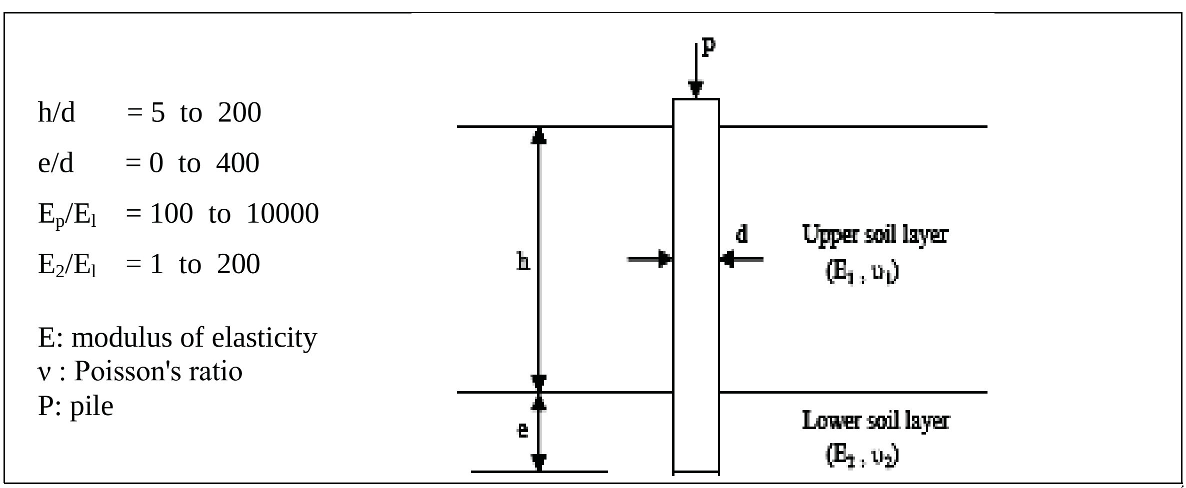

Figure 2 : Single pile problem embedded in two layers soil profile (after Chin, 2004)

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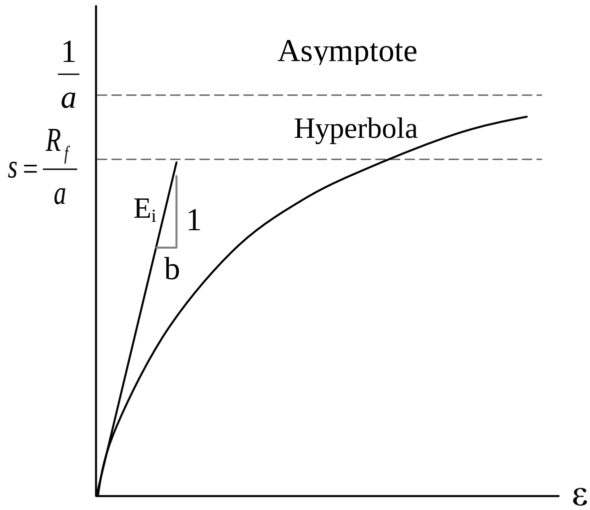

Figure 3 : Hyperbolic stress-strain curve for non-linear material (from Christian and Desai, 1977)

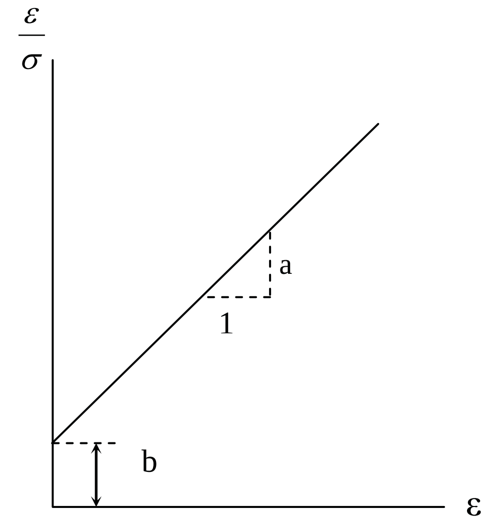

Figure 4 : Transformed hyperbolic stress-strain curve (from Christian and Desai, 1977)

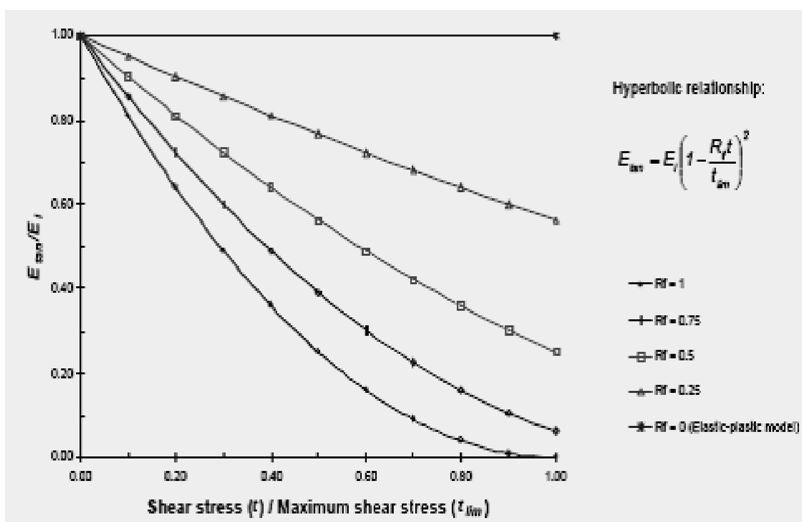

Figure 5 : Soil Young's modulus variation with stress level (from Geocentrix, 2002)

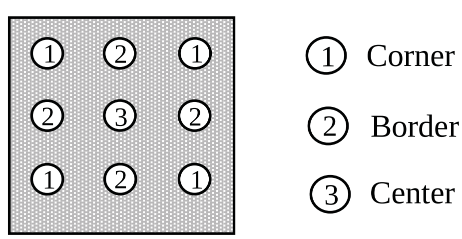

Figure 6 : Identifications of the 9 piles in the group

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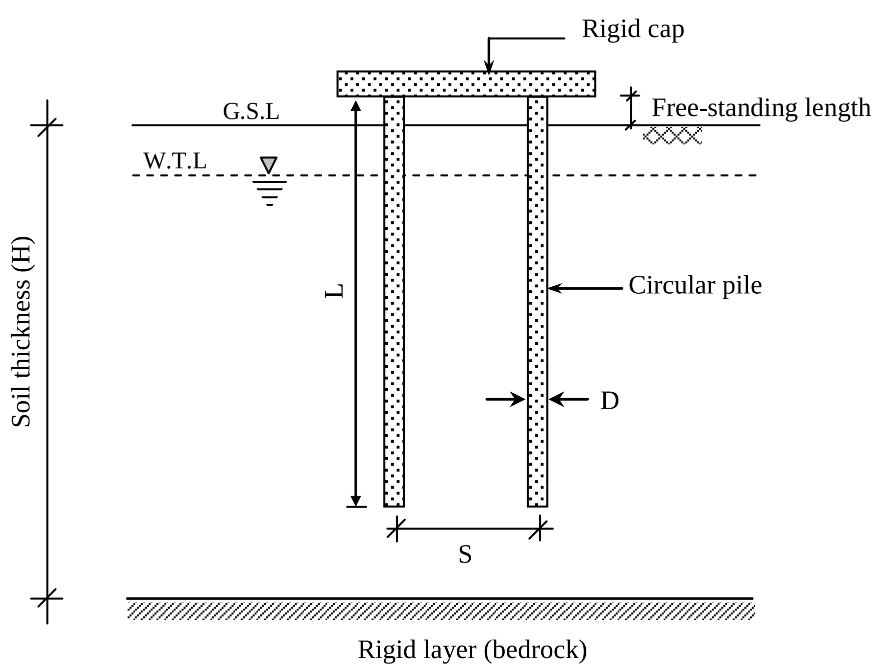

Figure 7 : The problem of pile group

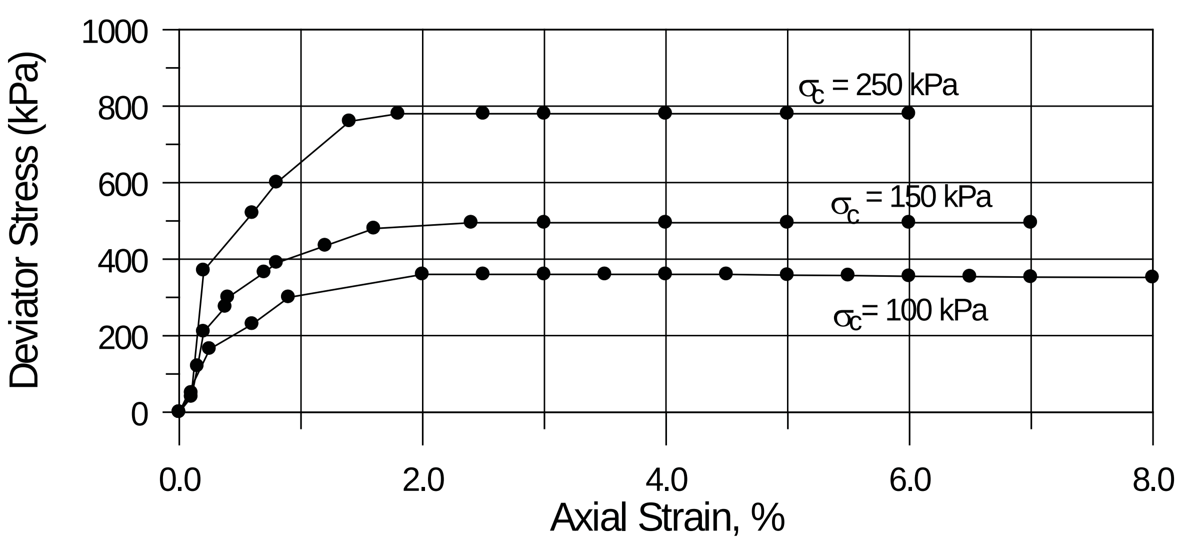

Figure 8 : Results of triaxial test (after Ghalib, 1975)

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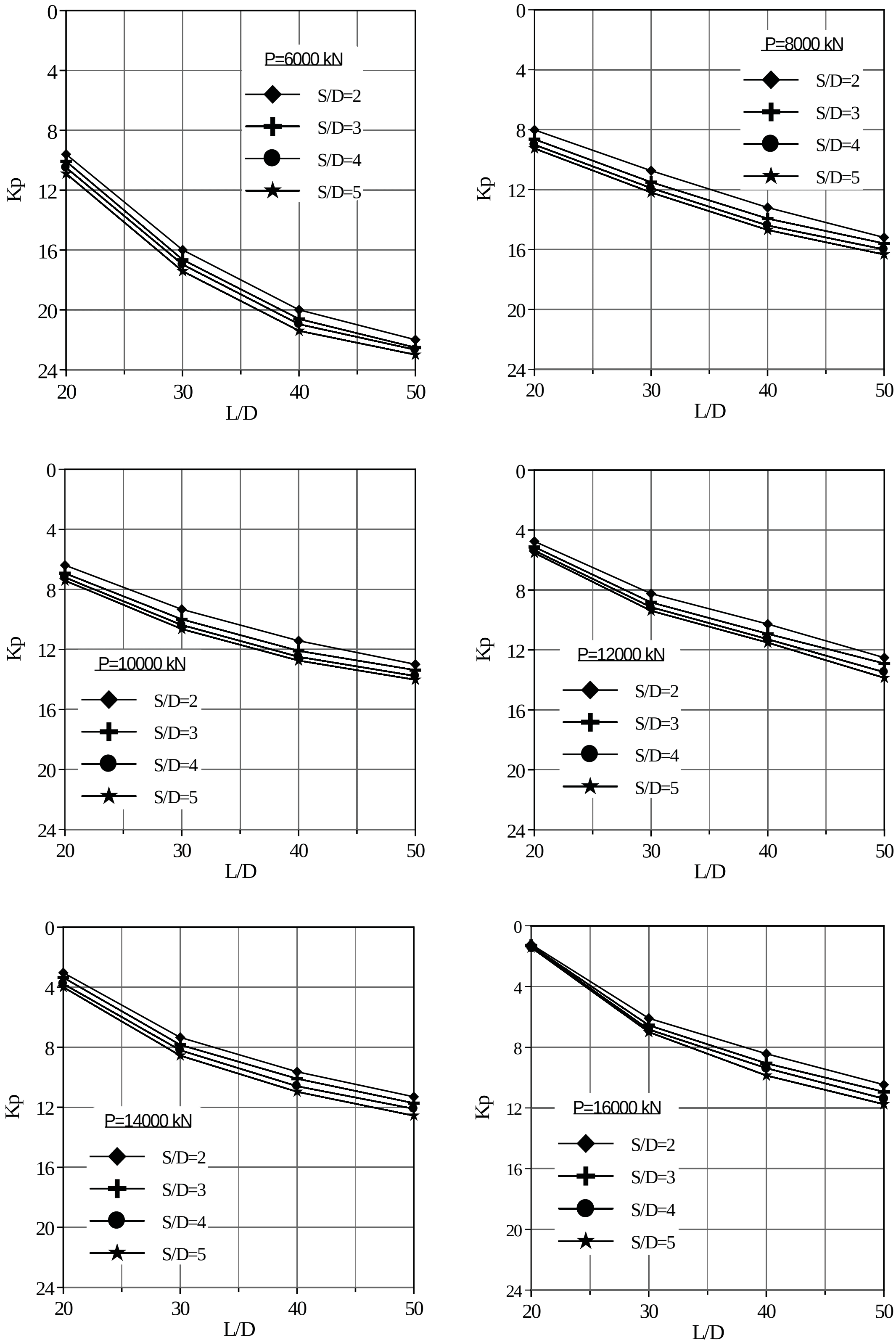

Figure 9 : Normalized load-settlement curves for (2*2) pile group of a diameter D=0.5 m, embedded in sand

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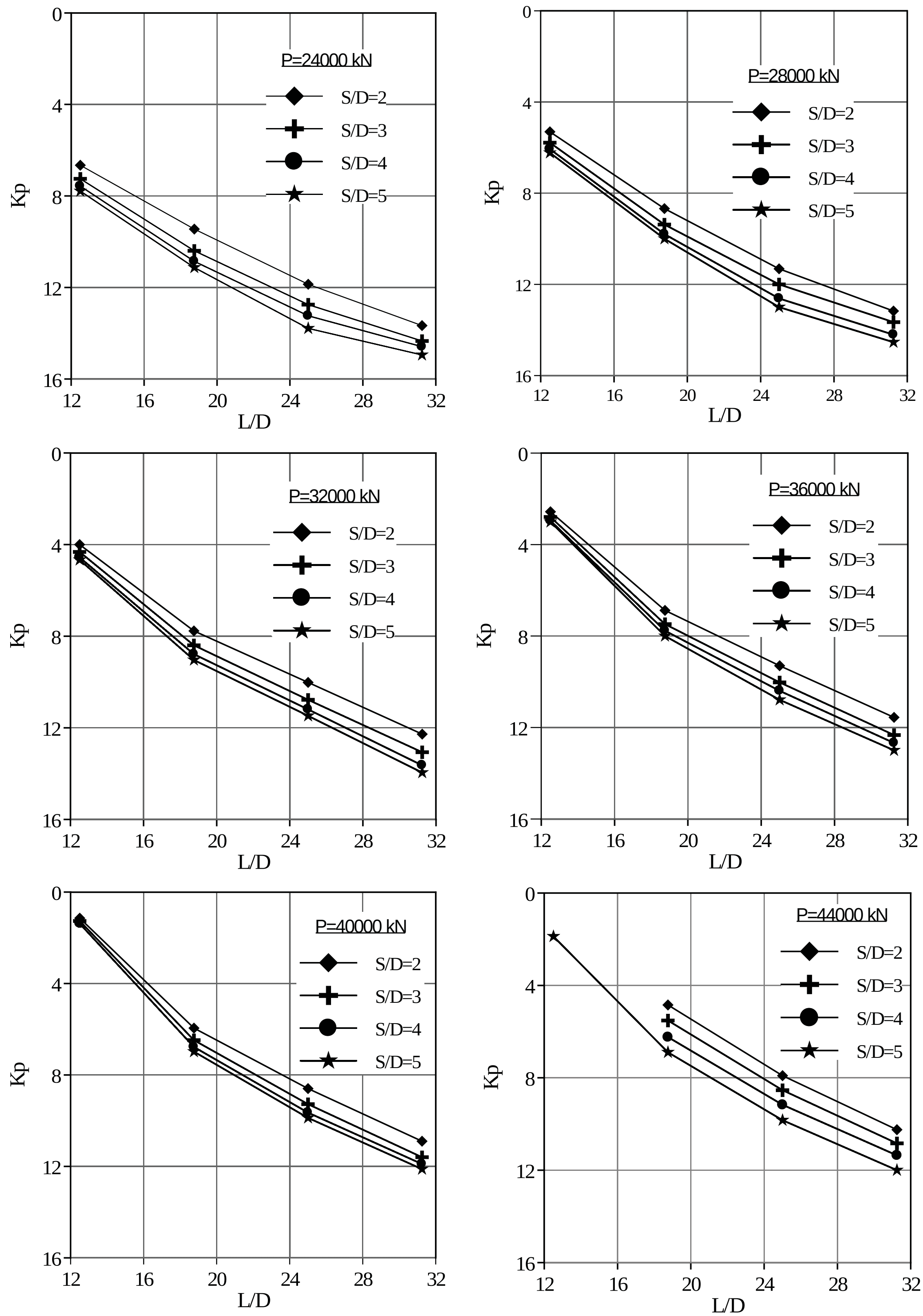

Figure 10 : Normalized load-settlement curves for (2*2) pile group of a diameter D=0.8 m, embedded in sand under different loadings

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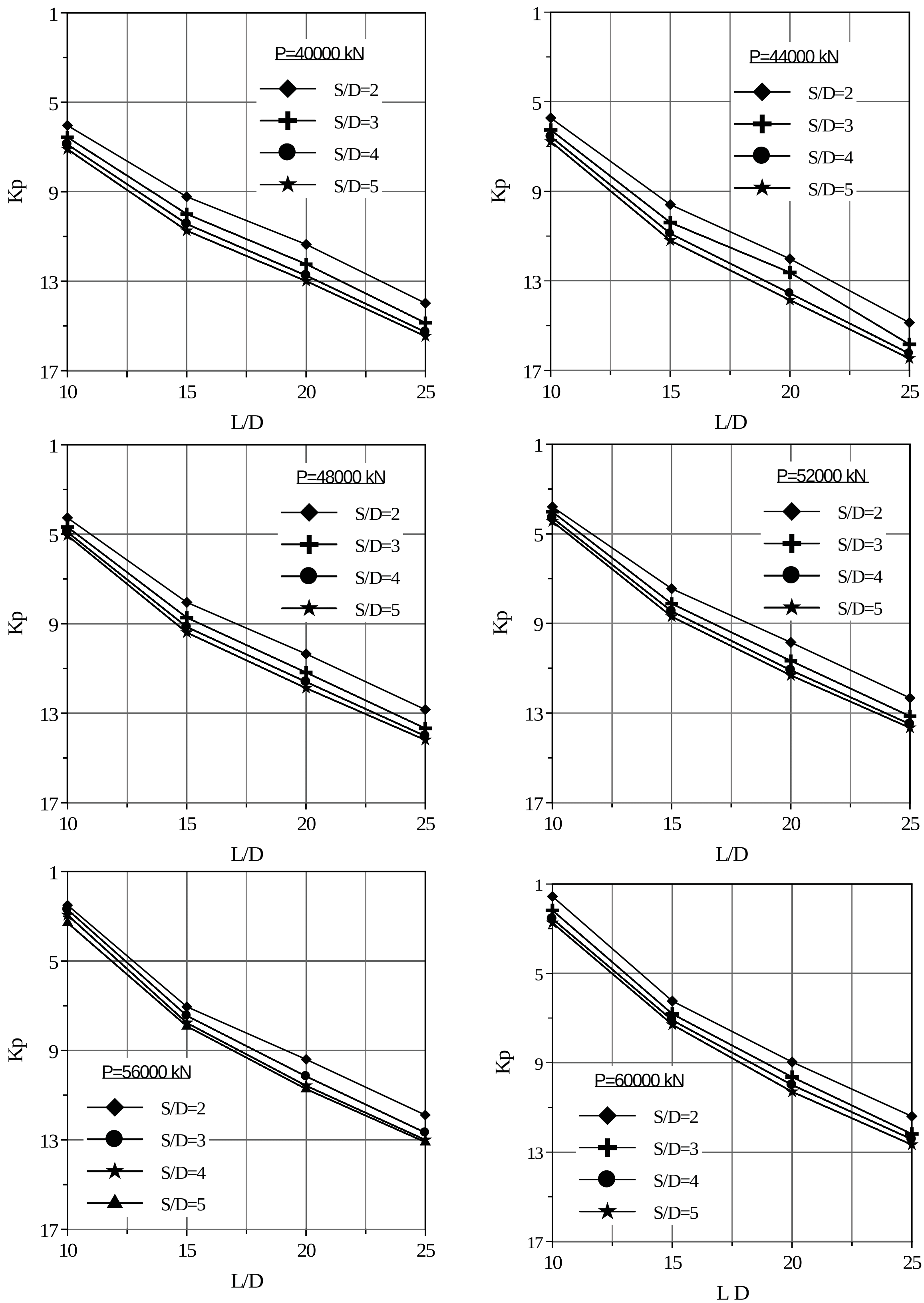

Figure 11: Normalized load-settlement curves for (2*2) pile group of a diameter D=1.0 m, embedded in sand under different loadings

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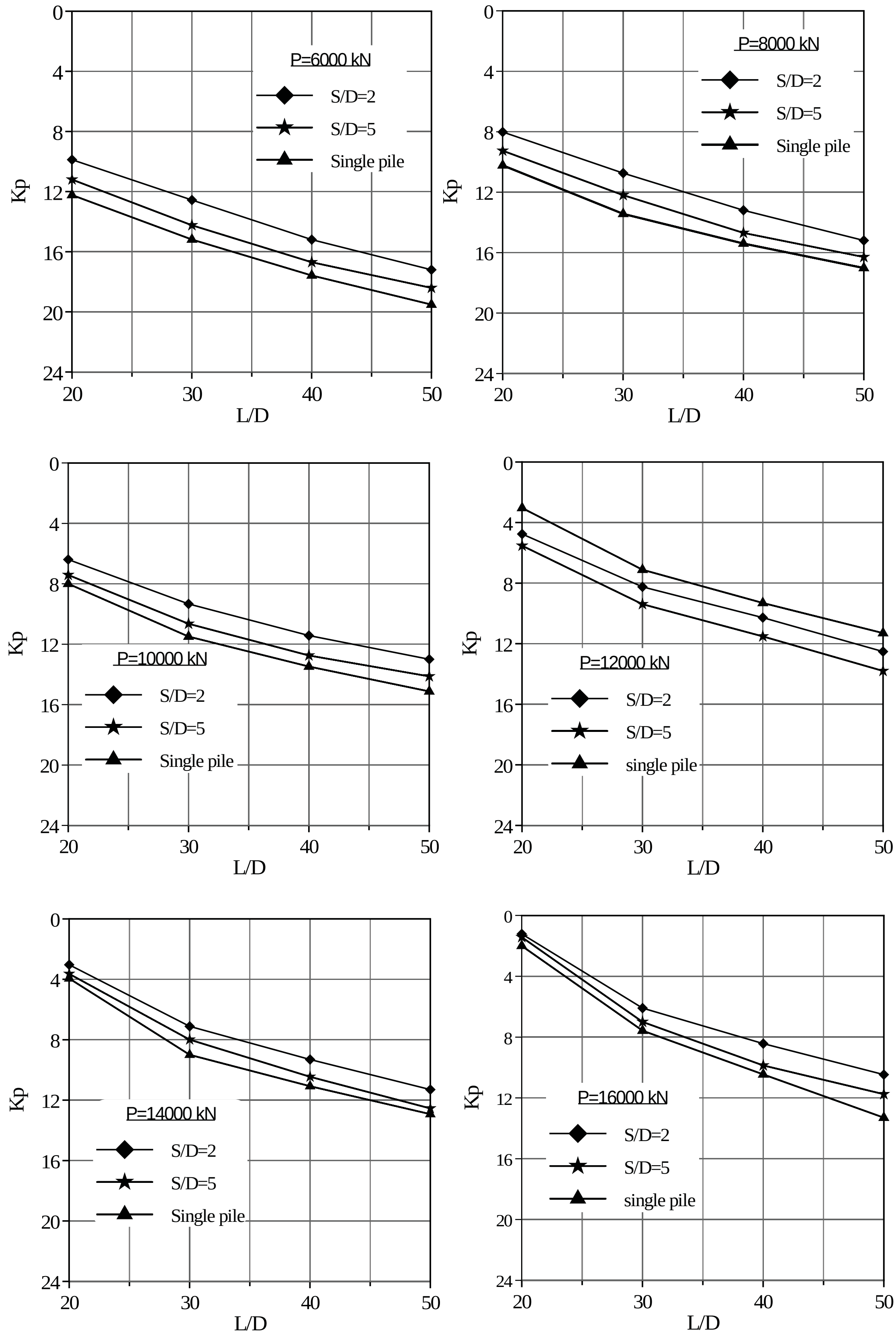

Figure 12 : Normalized load-settlement curves for (2*2) pile group of a diameter D=0.5 m, embedded in sand under different loadings as compared with a single pile

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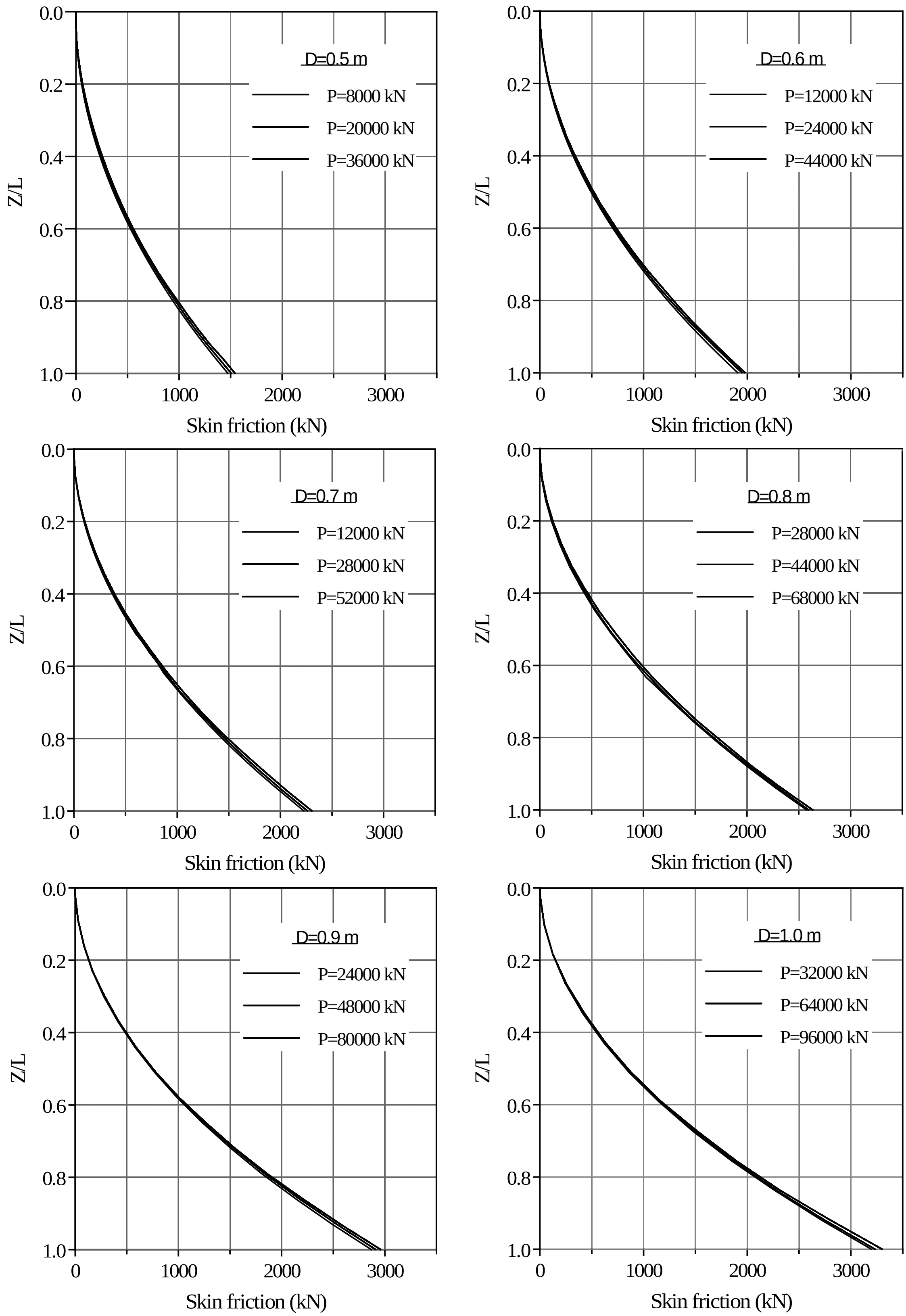

Figure 13 : Variation of skin friction along a single pile in (2*2) pile group embedded in sand for different diameters, (S/D=2, L=25 m)

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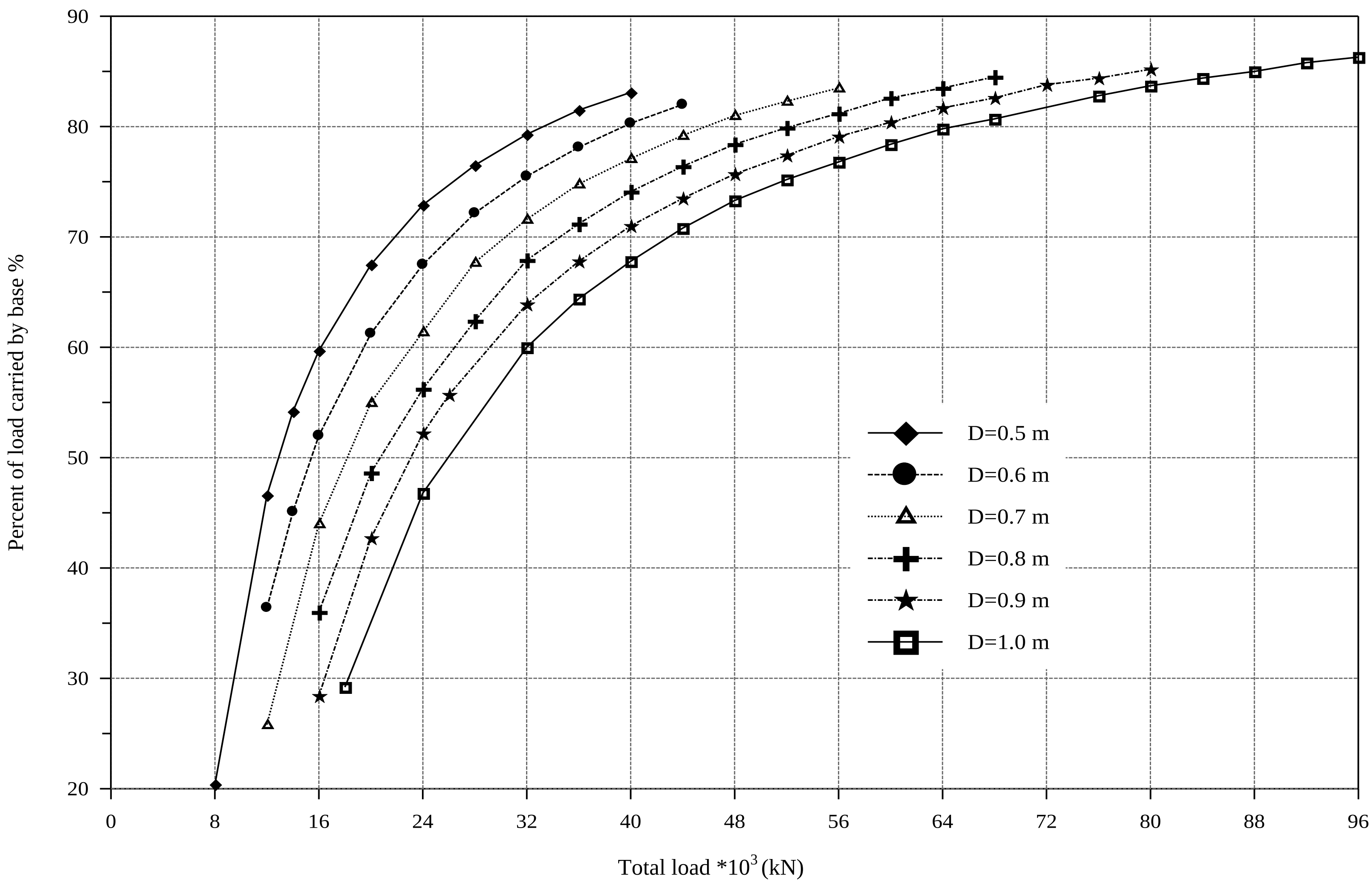

Figure 14 : Bearing force characteristics at th the pile base computed as a percentage of the applied load for a (2*2) pile group embedded in sand, (S/D=2, L=25 m)

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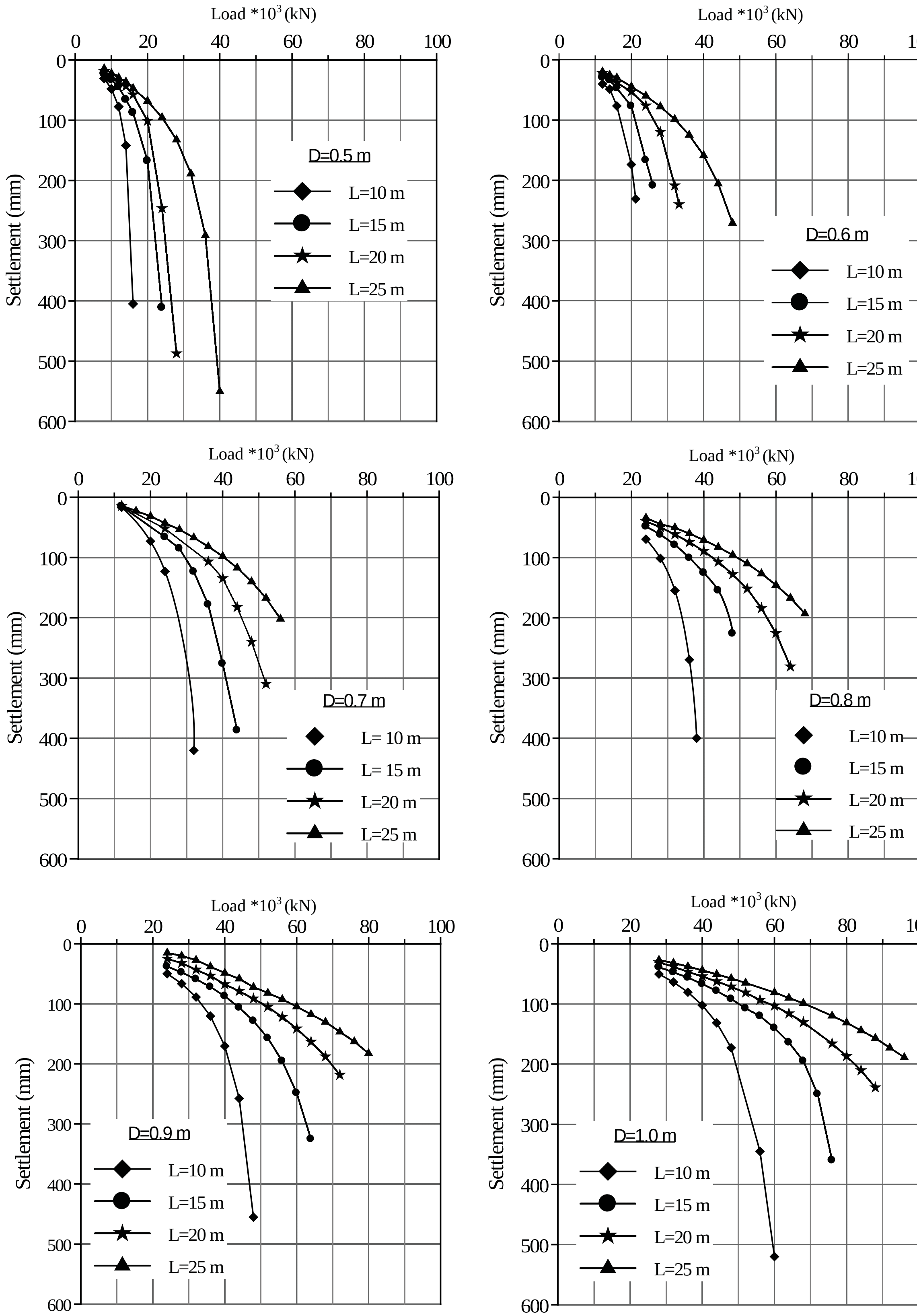

Figure 15 : Load-settlement curves for (2*2) pile group embedded in sand (S/D=2)

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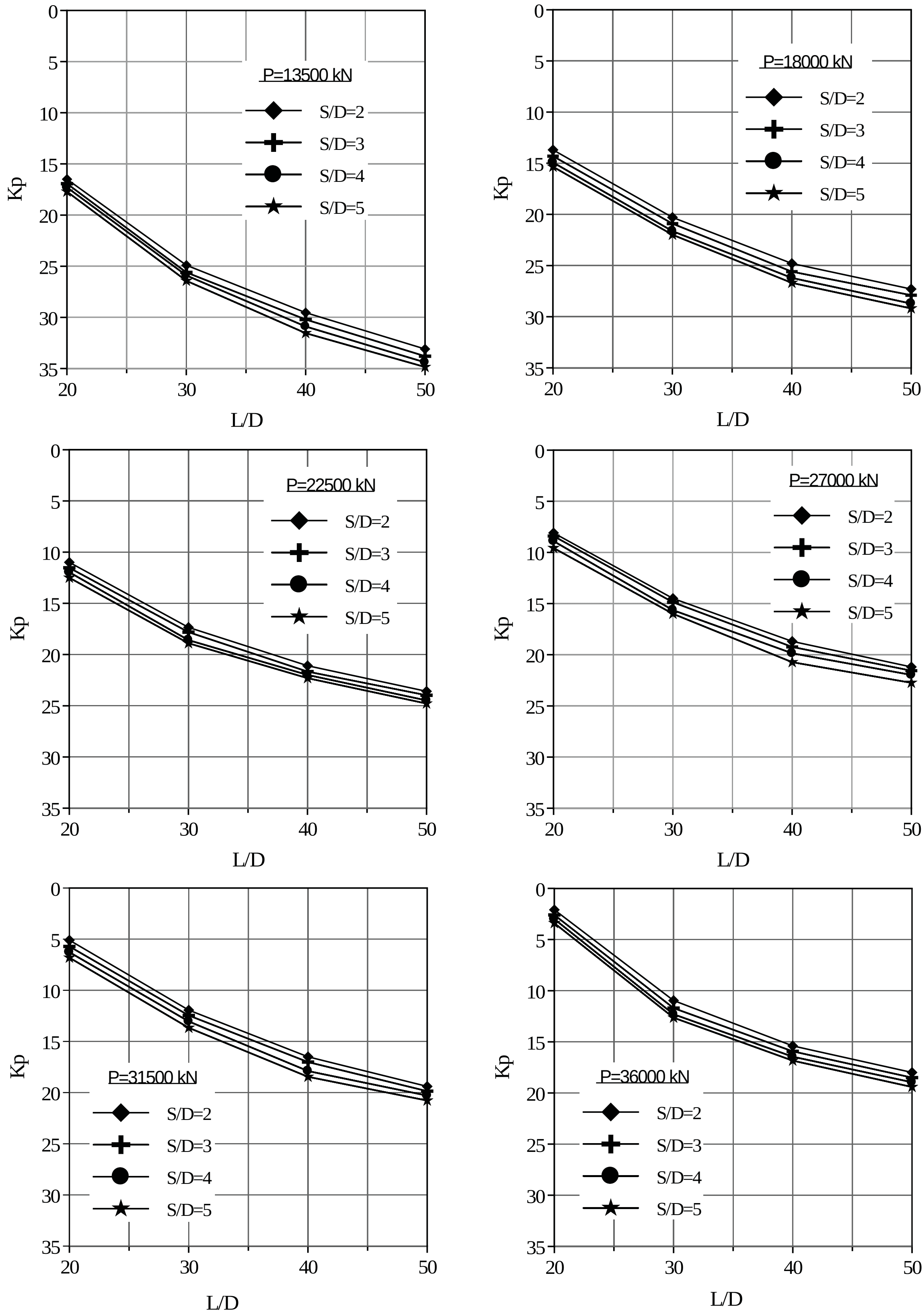

Figure 16 : Normalized load-settlement curves for (3*3) pile group of a diameter D=0.5 m, embedded in sand under different loadings

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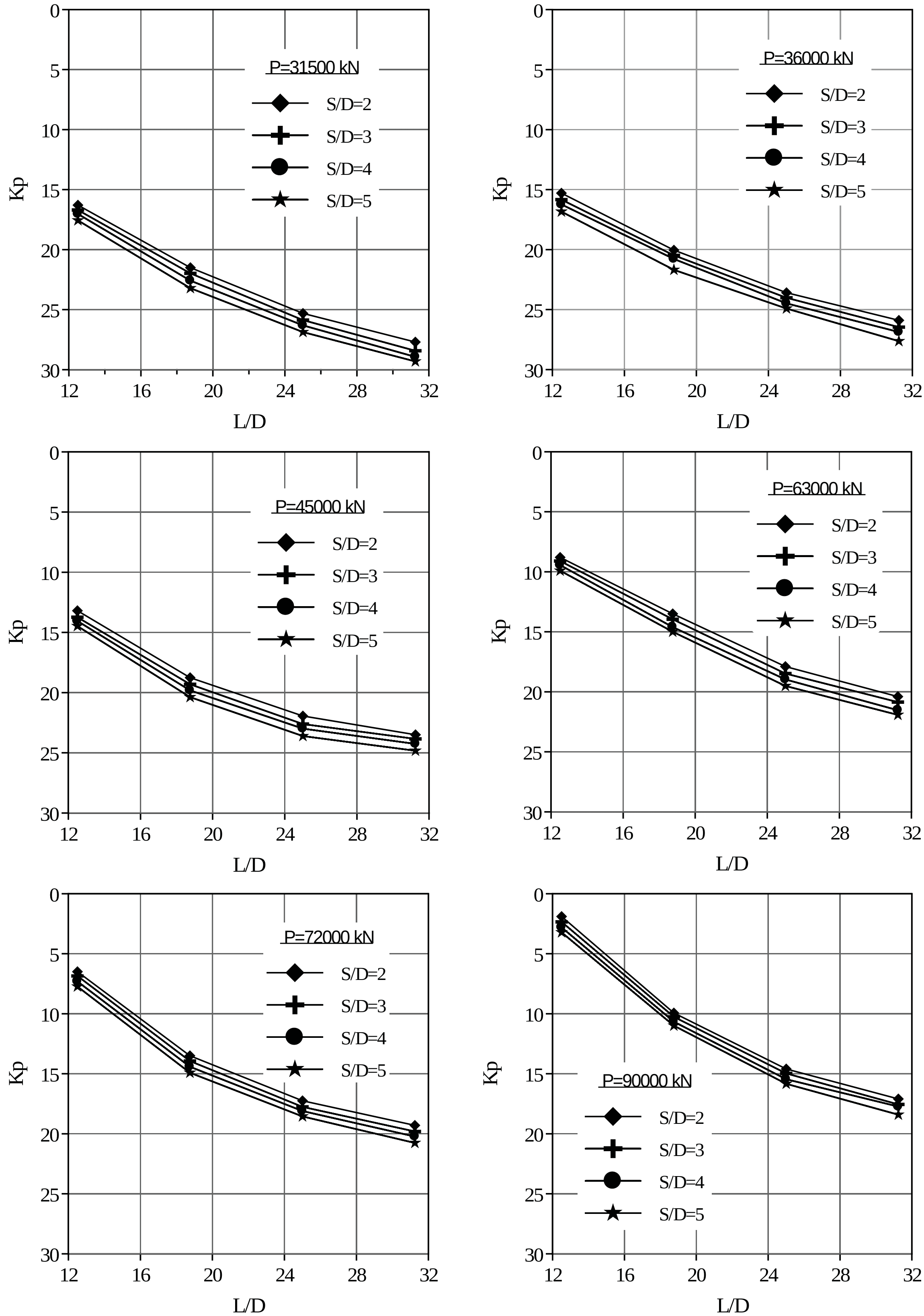

Figure 17 : Normalized load-settlement curves for (3*3) pile group of a diameter D=0.8 m, embedded in sand under different loadings

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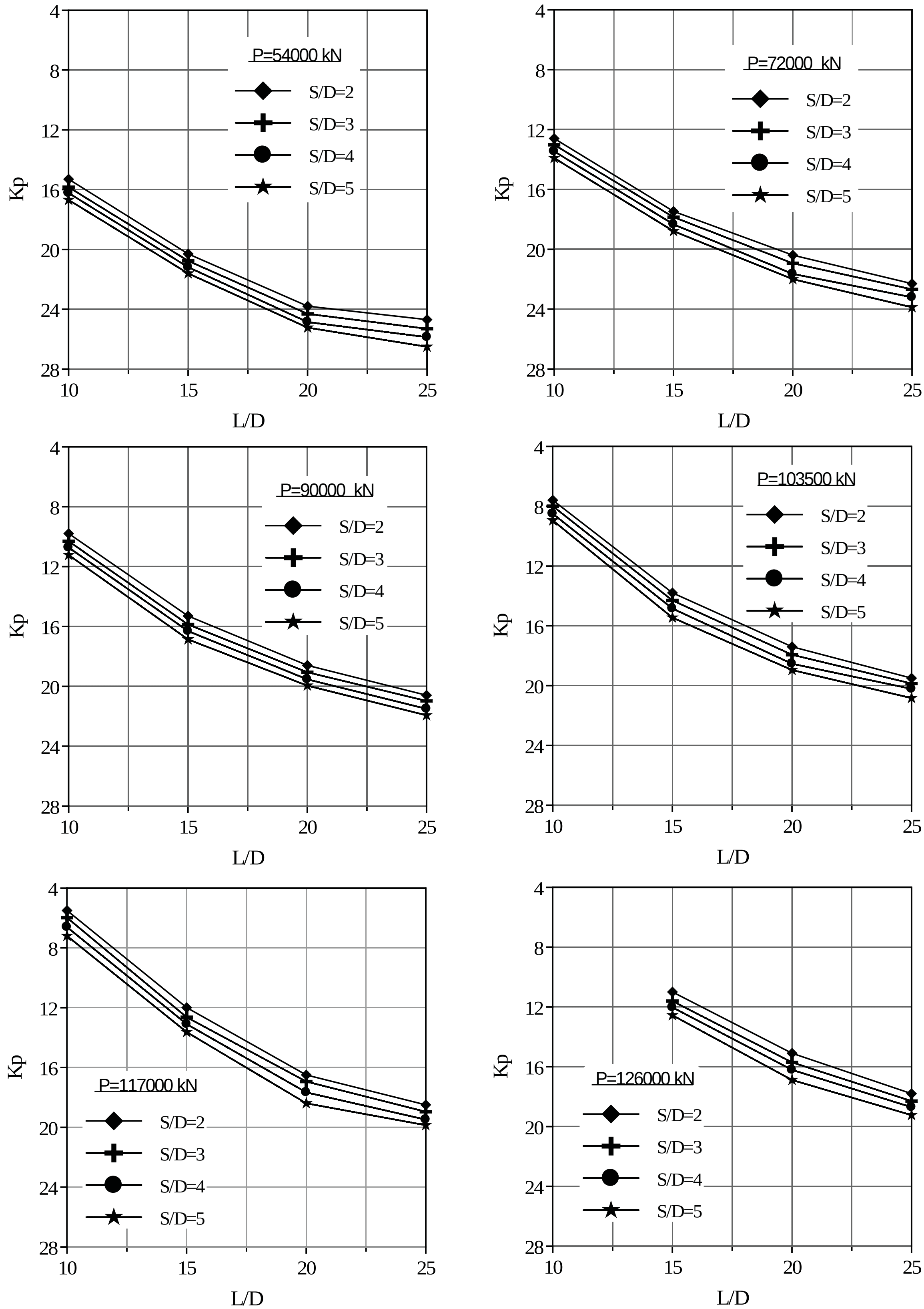

Figure 18 : Normalized load-settlement curves for (3*3) pile group of a diameter D=1.0 m, embedded in sand under different loadings

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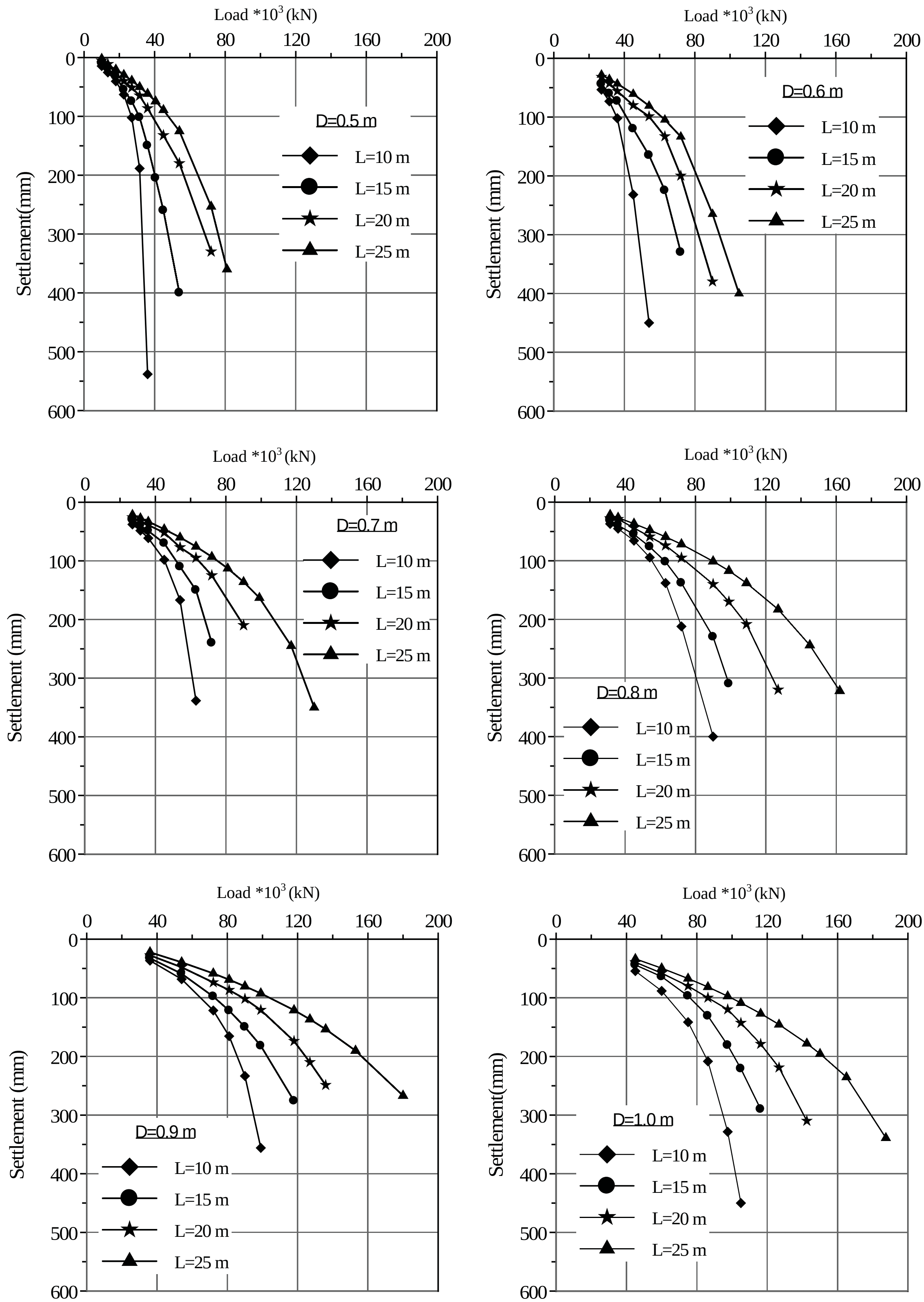

Figure 19 : Load-settlement curves for (3*3) pile group embedded in sand, (S/D=2)

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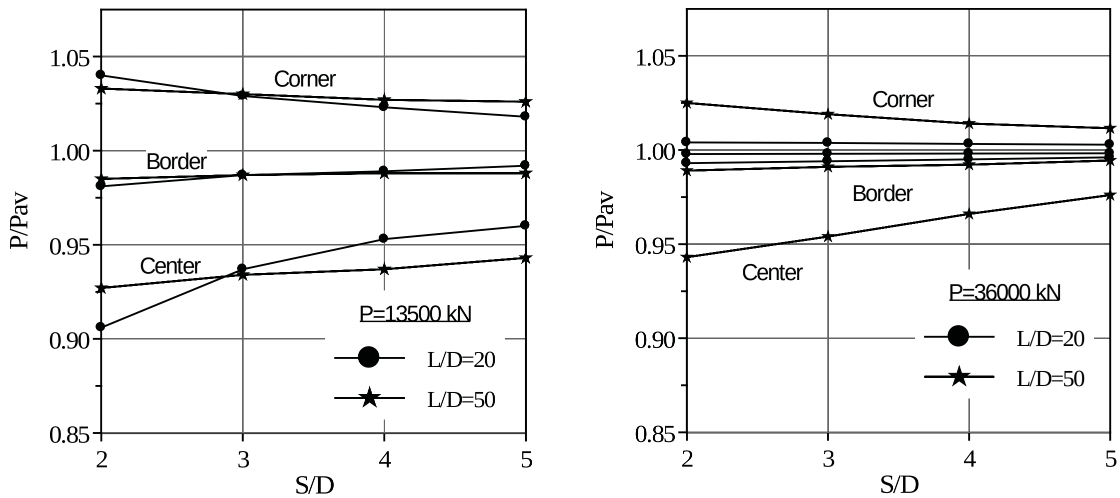

Figure 20 : Pile load distribution on (3*3) pile group embedded in sand, (D=0.5 m)

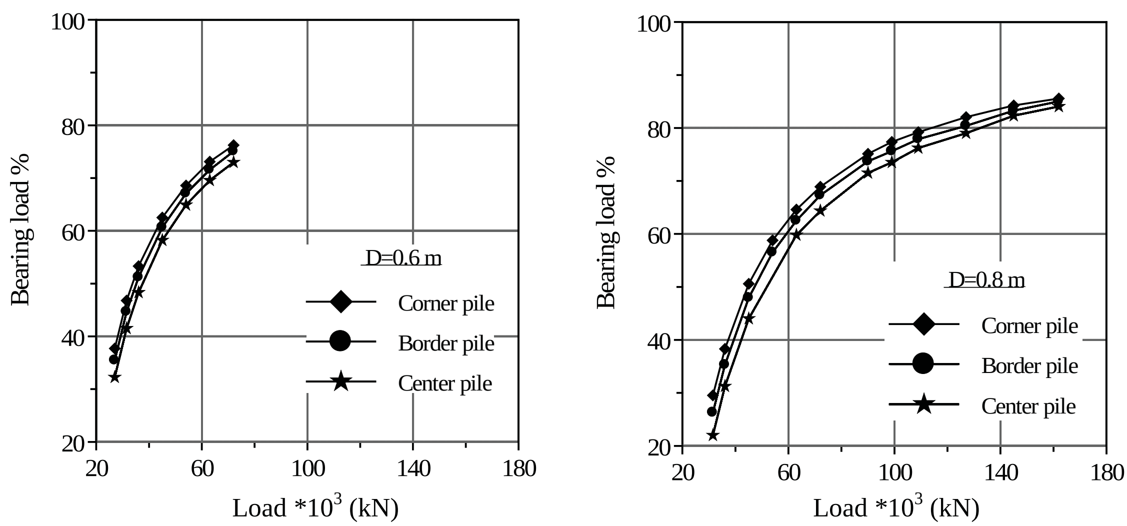

Figure 21 : Bearing force characteristics at thepile base computed as a percentage of the

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Identification of Appropriate Micromechanical Fracture Model for Predicting Fracture Performance of Steel Wires for Civil Engineering Applications

By Kazeem K Adewole
Newcastle University, United Kingdom

Abstract - The fracture performance of steel wires for civil,engineering applications remains a major concern in civil engineering construction and maintenance of wire reinforced structures. The need to employ approaches that simulate micromechanical material processes which characterizes fracture in civil structures has been emphasised recently in the literature. However, choosing from the numerous micromechanics-based fracture models, and identifying their applicability and reliability remains an issue that still needs to be addressed in a greater depth. Laboratory tensile testing and finite element tensile testing simulations with the shear, ductile and Gurson-Tvergaard-Needleman's micromechanicsbased models conducted in this work reveal that the shear fracture model is an appropriate fracture model to predict the fracture performance of steel wires used for civil engineering applications. The need to consider the capability of the micromechanics-based fracture model to predict the "cup and cone" fracture exhibited by the wire in choosing the appropriate fracture model is demonstrated.

Keywords : fracture performance, finite element, shear fracture model, ductile fracture model, gursontvergaard- needleman fracture model, wires.

GJRE Classification : FOR Code: 090599

IDENTIFICATION OF APPROPRIATE MICROMECHANICAL FRACTURE MODEL FOR PREDICTING FRACTURE PERFORMANCE OF STEEL WIRES FOR CIVIL ENGINEERING APPLICATIONS

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Identification of Appropriate Micromechanical Fracture Model for Predicting Fracture Performance of Steel Wires for Civil Engineering Applications

Kazeem K Adewole

Abstract - The fracture performance of steel wires for civil engineering applications remains a major concern in civil engineering construction and maintenance of wire reinforced structures. The need to employ approaches that simulate micromechanical material processes which characterizes fracture in civil structures has been emphasised recently in the literature. However, choosing from the numerous micromechanics-based fracture models, and identifying their applicability and reliability remains an issue that still needs to be addressed in a greater depth. Laboratory tensile testing and finite element tensile testing simulations with the shear, ductile and Gurson-Tvergaard-Needleman's micromechanics-based models conducted in this work reveal that the shear fracture model is an appropriate fracture model to predict the fracture performance of steel wires used for civil engineering applications. The need to consider the capability of the micromechanics-based fracture model to predict the "cup and cone" fracture exhibited by the wire in choosing the appropriate fracture model is demonstrated.

Keywords : fracture performance, finite element, shear fracture model, ductile fracture model, gurson-tvergaard-needleman fracture model, wires.

I. Introduction

Steel wires are used in civil engineering as pres-stressing steel wires and as suspension and/or cable-stayed bridge wires. They are also used as tensile armour wires which provide tensile and hoop reinforcement to flexible pipes that are used for offshore oil and gas transportation. The fracture performance (prediction of fracture load/stress, fracture strain/ displacement at fracture, fracture initiation point, fracture propagation and fracture path or sequence) of steel wires is a major concern in civil engineering construction and maintenance of civil engineering structures where wires provide the required structural reinforcement (Toribio and Ayaso, 2003). Recent research on the failure analysis of wires, such as the research conducted by Mahmoud, (2007) on bridge cable wires, and by Toribio and Valiente, (2004 and 2006) on concrete pre-stressing wires were based on experimental classical fracture mechanics approach using non-standardised fracture mechanics specimens. Non-standardised fracture mechanics specimens were used because the pre-stressing and suspended bridge wires are not large enough for standard traditional fracture mechanics test specimens to be manufactured from the wires [(Mahmoud, 2007); Toribio and Valiente, (2004 and 2006)]. The large specimen size requirement by the traditional classical fracture mechanics approach and the concern about the applicability of the traditional fracture mechanics in civil structures has necessitated the need to employ approaches that explicitly simulate micromechanical material processes which characterises fracture in civil structures (Fell and Kanvinde, 2009).

Micromechanics-based (micromechanical and phenomenological) fracture mechanics models serve as alternatives to the traditional classical fracture mechanics when standard fracture mechanics speci-mens cannot be obtained and when a safe use of the classical fracture mechanics concepts cannot be insured (Pardoen et al, 2010). Micromechanics fracture approach guarantees the transferability from specimens to structures over a wide range of sizes and geometries and is suitable for problems involving ductile fracture of crack-free bodies as it does not require the pre-cracked specimen needed for classical fracture mechanics tests (Bernauer and Brocks, 2002).

Micromechanical fracture modeling involves the modelling of void nucleation and growth, and is based on the assumption that ductile fracture occurs when the void volume fraction reaches a critical level; hence, such models involve modelling of void nucleation and growth (Dunand and Mohr, 2010). Phenomenological models are alternatives to micromechanical based models as they predict ductile fracture without modelling void nucleation and growth (Dunand and Mohr, 2010). Phenomenological models are based on the assumption that ductile fracture occurs when a weighted measure of the accumulated plastic strain, such as the equivalent plastic strain, reaches a critical value (Dunand and Mohr, 2010).

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The determination of some parameters for micromechanical fracture modelling can be done through metallurgical observations, while others require extensive and expensive material testing (Bernauer and Brocks, 2002). The parameters needed for phenomenological failure simulation, such as the modeling parameters for the shear and ductile failure simulations can be obtained experimentally. However, obtaining these parameters through direct experimentation may be difficult because it would require experiments over a range of stress triaxiality for the ductile failure simulation, and requires experiments over a range of shear stress ratio for shear failure simulation (Simulia, 2007). Consequently, the determination of the damage and failure parameters remains predominantly a phenomenological fitting procedure which requires a combination of testing and numerical simulations (Bernauer and Brocks, 2002). The phenomenological fitting procedure involves keeping some parameters constant and varying others during numerical simulations until the simulation results fit the experimental data, usually up to the fracture initiation point, which is marked by a sudden drop of load. The fracture initiation point represents the onset of macroscopic fracture at which void coalescence is "supposed" to start (Bernauer and Brocks, 2002). The values of the set of damage and fracture parameters at which the numerical data fits with the experimental data at the onset of macroscopic fracture has become a common technique to determine the critical fracture parameters (Bernauer and Brocks, 2002).

There are many micromechanical and phenomenological constitutive models for ductile damage and fracture prediction. However, choosing from the numerous micromechanical and phenomenological models, and identifying their applicability and reliability remains an issue that still needs to be addressed in greater depth (Li et al, 2011). This is because using an inappropriate model may result in unreliable or inappropriate ductile fracture predictions, which has been a problematic issue in industrial applications of ductile failure models (Li et al, 2011). Also the need to identify the ductile fracture model which is able to predict ductile fracture in a way that is to the largest extent in agreement with actual phenomena in a material has been stressed by (Rakin et al, 2004). However, in most published literature such as Bernauer and Brocks, (2002); Dunand and Mohr, (2010); Li et al, (2011) and Rakin et al, (2004), the appropriateness (applicability and reliability) of many ductile fracture models to describe the fracture behaviour of materials are based on the ability of such models to predict force-displacement/reduction in area curves that agree with the experimental curve up to the fracture initiation point. The simulation techniques employed to obtain such curves have also been adjudged to be appropriate without any consideration for the ability of the models to predict the actual fracture shape(s) (actual phenomena) exhibited by the materials/components/specimens.

In this work, the identification of the appropriate ductile fracture model for a typical high strength steel wires used in civil and structural engineering applications from three micromechanics-based models (two phenomenological fracture models: shear and ductile fracture models; and Gurson-Tvergaard-Needleman's micromechanical model) that are inbuilt in Abaqus 6.9-1 finite element (FE) code was conducted by comparing the force-displacement curves and the fracture shapes obtained from experimental tensile testing and finite element (FE) tensile testing simulations. The simulations with the three micromechanics-based fracture models were conducted with the isotropic elastic-plasticity model in-built in Abaqus 6.9-1 finite element code. Details of the isotropic elastic-plasticity model and the three micromechanics-based fracture models are presented in the following sections.

a) Isotropic elastic-plasticity model

The isotropic elastic-plasticity model in Abaqus is based on linear isotropic elasticity theory and a uniaxial-stress, plastic-strain strain-rate relationship (Simulia, 2007). The elastic aspect of the model is defined in terms of its volumetric and deviatoric components given in equations 1 and 2 respectively obtained from Simulia, (2007). The model is based on a von Mises yield surface with the yield function, , given in equation 3 and a flow rule given in equation 4 obtained from Simulia, (2007).

$$p = -K\varepsilon_{vol} \tag{1}$$

$$S = 2Ge^{el} \tag{2}$$

$$f = q = \sqrt{\frac{3}{2}S : S} \tag{3}$$

$$de^{pl} = d\bar{e}^{pl} n \tag{4}$$

Where p is the hydrostatic pressure, ε_{vol} is the volume strain, S is the deviatoric stress e^{el}, is the deviatoric elastic strain, q is the von Mises equivalent stress, e^{pl} is the deviatoric plastic strain, $\bar{e}^{pl}$ is the equivalent plastic strain, $n = \frac{3}{2}\frac{S}{q}$ K, is the bulk modulus and G is the shear modulus. K and G are calculated from the Young's modulus, E, and Poisson's ratio, ν, of the material.

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b) Shear Failure Model

The shear failure criterion is a phenomenological model for predicting the onset of damage due to shear bands. Applied stress causes shear band formation and localisation, leading to the formation of cracks within the shear bands and eventual failure (Hooputra et al, (2004). The shear model assumes that the equivalent fracture strain is a function of the variable given in equation 5 obtained from Hooputra et al, (2004).

$$\theta = \frac{1 - k_s \eta}{\phi} \quad (5)$$

Where k_S is a material parameter, η is the stress triaxiality and ϕ is the ration of the maximum shear stress and the equivalent stress (von Mises) as given in equation 6 obtained from Hooputra et al, (2004).

$$\phi = \frac{\tau_{\max}}{\sigma_{eq}} \quad (6)$$

The equivalent plastic strain for shear fracture with respect to θ is given in equation 7 obtained from Hooputra et al, (2004).

$$\varepsilon_{eq}^{**} = \frac{\varepsilon_S^{+} \sinh\left[f(\theta^{+} - \theta^{-})\right] + \varepsilon_S^{-} \sinh\left[f(\theta^{+} - \theta^{-})\ \right]}{\sinh\left[f(\theta^{+} - \theta^{-})\ \right]} \quad (7)$$

θ^{+} and θ^{-} are the values of the parameter θ for equibiaxial tension and compression, ε_S^{+} and ε_S^{-} are the equivalent plastic strain in equibiaxial tension/compression at shear fracture and f is an orientation dependent parameter.

c) Ductile damage and fracture criterion

The ductile damage criterion is a phenomenological model for predicting the onset of damage by micro-void nucleation, void growth and void coalescence. Micro-void nucleation could be as a result of micro-cracking of particles and/or fracture or decohesion of second phase inclusions. Plastic straining causes the nucleated voids to grow or enlarge, leading to localisation of plastic flow between the enlarged voids and eventual ductile tearing of the ligaments between the enlarged voids [14]. The ductile failure model assumes that the equivalent fracture strain ε_{eq}^{**} given in equation 8 is a function of stress triaxiality η and the equivalent plastic strain at ductile fracture (Hooputra et al, 2004).

$$\varepsilon_{eq}^{**} = \frac{\varepsilon_T^{+} \sinh\left[c(\eta^{-} - \eta)\right] + \varepsilon_T^{-} \sinh\left[c(\eta - \eta^{+})\ \right]}{\sinh\left[c(\eta^{-} - \eta^{+})\ \right]} \quad (8)$$

Where ε_T^{+} and ε_T^{-} are the equivalent plastic strain in equibiaxial tension/compression at ductile fracture, and η^{+} and η^{-} are the stress triaxiality in equibiaxial tension / compression at ductile fracture and is a material parameter.

d) Porous metal plasticity

The porous metal plasticity (PMP) model is a micromechanical model used in modelling damage and failure of voided metals. It is based on the Gurson's porous metal plasticity theory which is based on the assumption that the yield stress of the fully dense matrix material is a function of the equivalent plastic strain in the matrix. It predicts failure by nucleation of new voids, and growth and coalescence of both existing voids and nucleated voids, with final failure occurring by ductile crack propagation (ductile tearing). The Gurson's porous metal plasticity model yield condition modified by Tvergaard, (1981) is given in equation 10.

$$\phi = \left(\frac{q}{\sigma_y}\right)^2 + 2q_1 f^* \cosh\left(-q_2 \frac{3p}{2\sigma_y}\right) - \left(1 + q_3 f^{*2}\right) = 0 \quad (10)$$

where:

$$q = \sqrt{\frac{3}{2} S : S}$$

$$S = pI + \sigma,$$

$$p = -\frac{1}{3}\sigma : I$$

I is the identity tensor, σ is the Cauchy stress tensor, σ_y is the yield stress, q_1 and q_2 are the coefficients of the void volume fraction, and q_3 is the coefficient of pressure term. The function f^* models the rapid loss of stress carrying capacity that accompanies void coalescence and is defined in terms of the void volume fraction, f, in equations 11 to 13

$$f^* = f \qquad \text{if} \qquad f \le f_c \quad (11)$$

$$f^* = f_c + \frac{f_F - f_c}{f_F - f_c}(f - f_c) \qquad \text{if} \qquad f_c \le f \le f_F \quad (12)$$

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$$f^* = \bar{f}_F \qquad \text{if} \qquad f \geq f_F \qquad (13)$$

where:

$$\bar{f}_F = \frac{q_1 + \sqrt{q_1^2 - q_3}}{q_3}$$

f_c Critical value of the void volume fraction at which void nucleation begins,

f_F Value of void volume fraction at which there is a complete loss of stress carrying capacity in the material (failure)

Material failure occurs when $f_c < f < f_F$, due to mechanisms such as micro fracture and void coalescence. Total failure at the material point occurs when $f \geq f_F$, leading to the removal of the elements (Tvergaard, 1981).

II. Experimental

The details of the experimental measurements and FE simulations are presented in this section.

a) *Laboratory tensile testing*

Un-machined full cross section specimens of 12mmx5mm and 12mmx7mm wires sizes recommended by ASTM E8M: 2009 and BS EN 10002-1:2001 were tested with an Instron universal testing machine (IX 4505) with a maximum static capacity of ±100 kN. The Instron universal testing machine was fitted with an Instron 2518 series load cell and the displacement was measured using an Instron 2630-112 clip-on strain gauge extensometer with a 50 mm gauge length.

b) *Finite element tensile testing simulation*

The three dimensional FE simulation of the tensile testing of the wire specimens was conducted using the in-built isotropic elastic-plastic model combined with the ductile, shear and porous metal plasticity fracture models in Abaqus 6.9-1 finite element code. The FE simulation was conducted by fixing the left hand end of the full three dimensional model of the wire and subjecting the right hand end, which is free to move only in the direction of the tensile load to a longitudinal displacement as shown in Figure 1. The FE tensile testing simulation was conducted for the 12mm x 5mm and 12mm x 7mm wire sizes. The outer regions of the model of the wires specimen were meshed with 1mmx1mmx1mm C3D8R elements(8-node hexahedral linear brick reduced integration elements with hourglass control) and the middle region of the model of the wire specimen was meshed with refined elements with 0.25x0.25x1mm dimension as shown in Figure 1. 0.25x0.25x1mm element size with the 1mm dimension in the direction of the length of the specimen was established through mesh convergence study to be the optimum element size required for accurate predictions of the wire's force-displacement response and fracture shape.

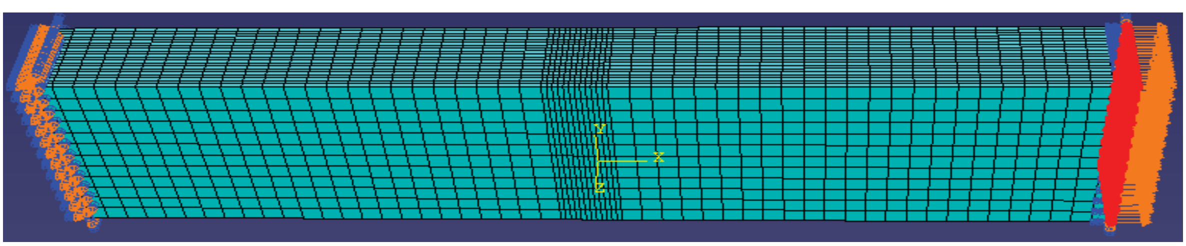

Figure 1 : Meshed model of wire specimen

a) *Calibration of the modeling parameters*

The calibrated modeling parameters employed for the simulations conducted with the three ductile fracture models were obtained using the phenomenological curve fitting procedure described earlier in section 1.0. Simulations were conducted with varying model parameter combinations, a few of the parameter combinations presented in this paper designated as parameter combinations A to D, E to H and I to L for the shear, ductile and PMP models respectively are presented in Tables 1, 2 and 3 respectively. Parameter combinations A, E and I which were the starting parameter values are typical parameters for ductile materials. The parameters were varied until the FE predicted force-displacement curves with the same fracture point as the experimental curve.

For the PMP model, the coefficients of the void volume fraction and were fixed at 1.5 and 1.0 respectively, while the coefficient of pressure term, the average nucleation strain and the standard deviation were fixed at 2.25, 0.3 and 0.1 respectively for all the simulations.

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Table 1 : Shear fracture parameter combinations investigated

Parameter combinations	Fracture strain	Shear stress ratio	Strain rate (s^{-1})	Parameter Ks
Parameters combination A	0.2761	10	0.0001	0.3
Parameters combination B	0.345125	12.5	0.000125	0.3
Parameters combination C	0.41415	15	0.00015	0.3
Parameters combination D	0.5522	20	0.0002	0.3

Table 2 : Ductile fracture parameter combinations investigated

	Fracture strain	Stress triaxaility	Strain rate (s^{-1})
Parameters combination E	33.238	3.3333	0.0001
Parameters combination F	36.5618	3.66663	0.00011
Parameters combination G	49.857	4.99995	0.00015
Parameters combination H	66.476	6.6666	0.0002

Table 3 : Porous metal plasticity parameter combinations investigated

	Void volume fraction f_N	Critical void volume fraction at failure f_c	Total void volume fraction at failure f_F
Parameters combination I	0.01	0.01	0.15
Parameters combination J	0.001	0.001	0.015
Parameters combination K	0.002	0.002	0.03
Parameters combination L	0.004	0.004	0.06

III. Results

The normalized force-displacement curves (normalised with the experimental ultimate load and fracture point displacement values for confidentiality purposes) predicted by the simulations of the tensile testing of the 12mmx5mm wire specimens with varying shear, ductile and PMP model parameter combinations are shown in Figures 2,3 and 4 respectively. As shown in Figures 2-4 and as presented in Figure 5, the simulations with the shear parameters combination B, ductile parameters combination F and PMP parameters combination L predicted fracture points are the closest to the experimental fracture point. The phenomenological fitting to calibrate the damage parameters for the 12mmx7mm wire was conducted in a similar manner and the force-displacement curves obtained from the parameter combinations at which the FE simulations predicted force-displacement curves with approximately the same fracture point as the experimental curve are shown in Figure 6.

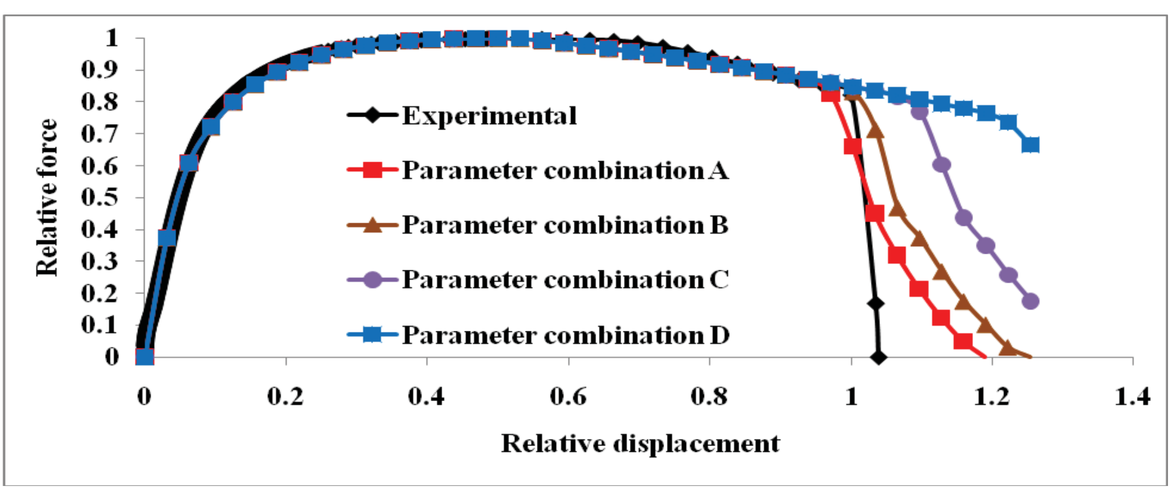

Figure 2 : Force-displacement curves from simulations with varying shear failure modelling parameter combinations

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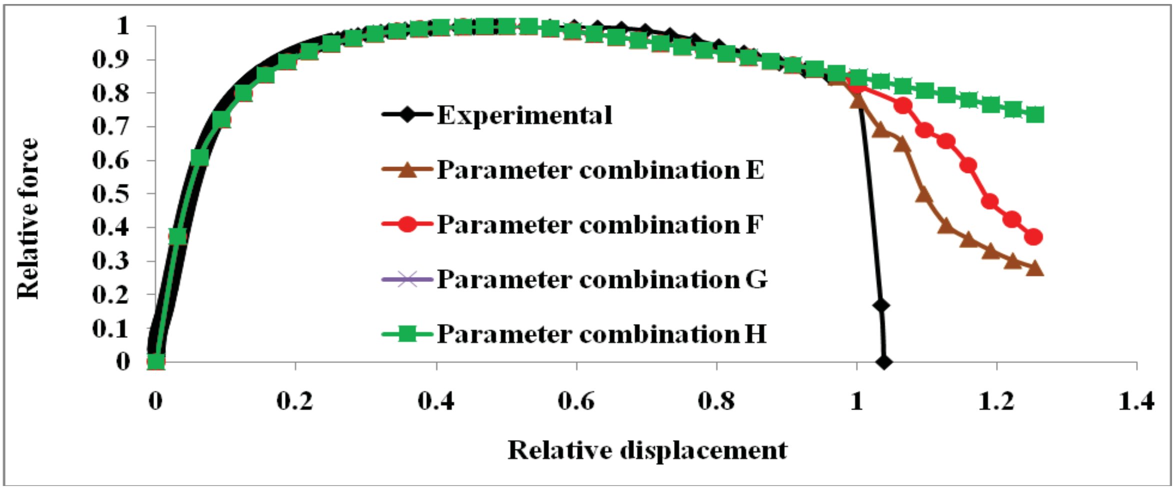

Figure 3 : Force-displacement curves from simulations with varying ductile failure modelling parameter combinations

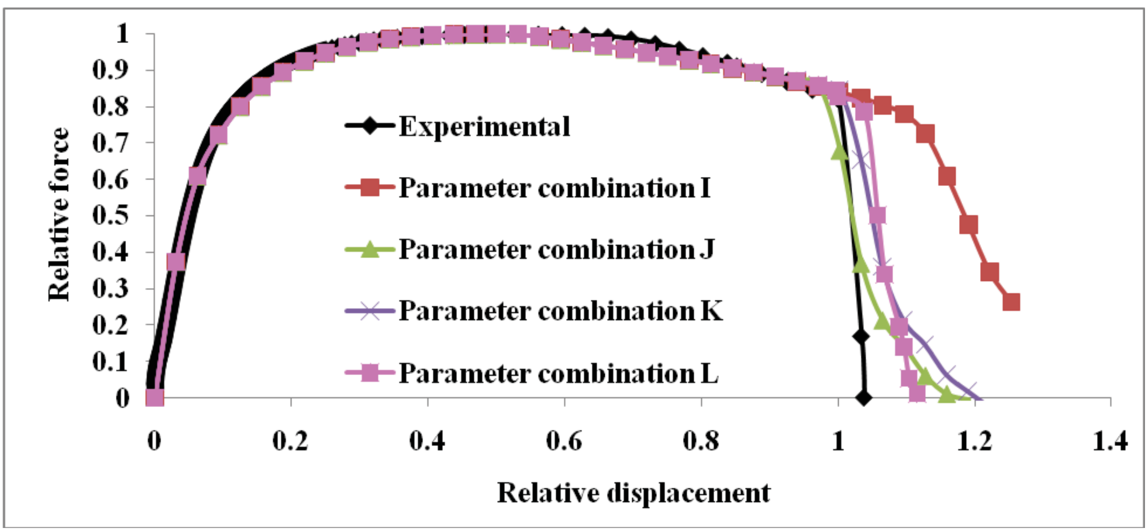

Figure 4 : Force-displacement curves from simulations with varying PMP failure modelling parameter combinations

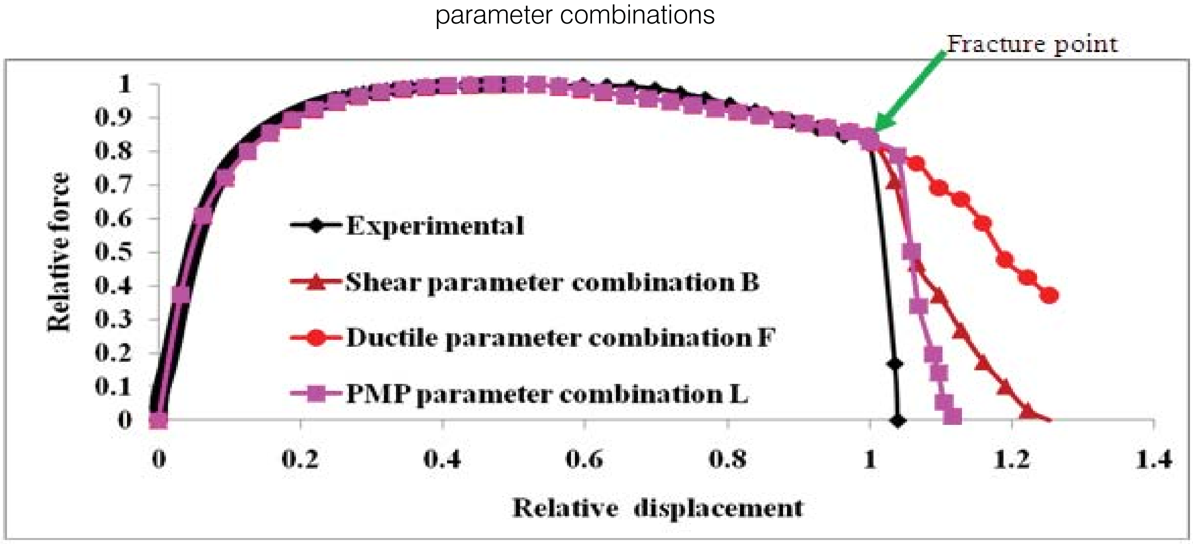

Figure 5 : Force-displacement curves from experiment and FE with calibrated shear, ductile and PMP failure modeling parameters for 12mm x 5mm wire

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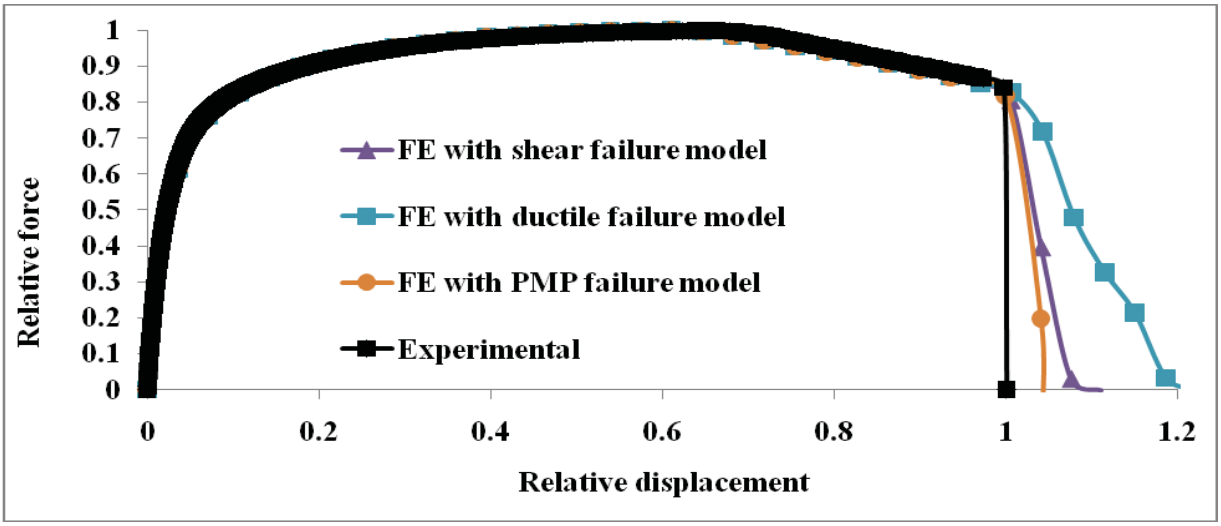

Figure 6 : Force-displacement curves from experiment and FE with calibrated shear, ductile and PMP failure modeling parameters for 12mm x 7mm wire

The fractured laboratory specimens of the two wire sizes which exhibit a “cup and cone” fracture (flat fracture at the center and slant fracture at the outer regions of the specimen) are shown in Figure 7. The fracture shapes predicted by the simulations conducted with the varying shear, ductile and PMP model parameter combinations for the two wire sizes are the same. Consequently, only the fracture shapes predicted by the simulations conducted with the porous metal plasticity, ductile and shear failure models with modeling parameters that predicted force-displacement curves with approximately the same fracture point as the experimental curve for the 12mm x 5mm wire are shown in Figures 8a, b and c. The cup and cone fracture shape predicted by the simulation conducted with the shear failure models for the 12mm x 7mm wire are also presented in Figure 8d.

(a) 12mmx5mm wire

(b) 12mmx7mm wire

Figure 7 : Cup and cone fracture shape exhibited by the laboratory tensile tested wire specimens

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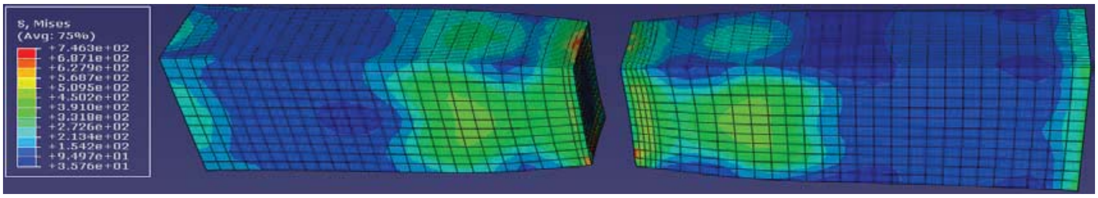
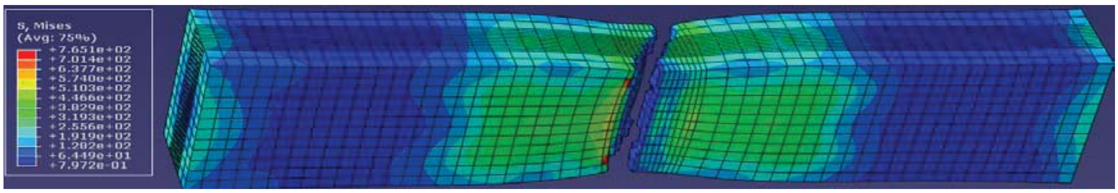
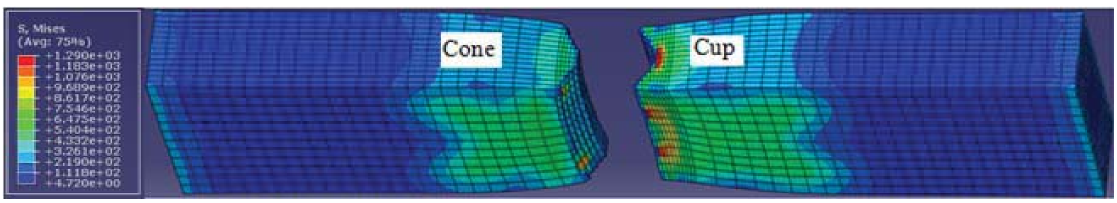


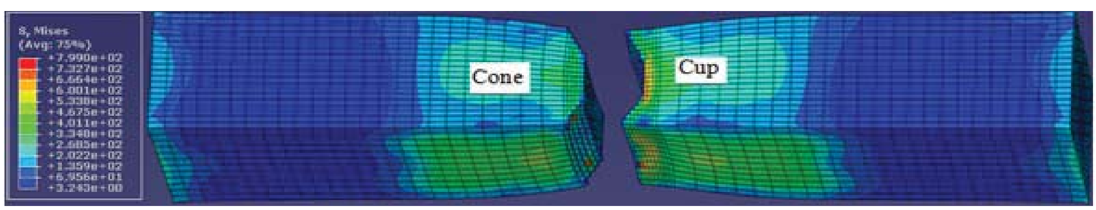
(a) Fracture shape predicted by FE simulation with porous metal plasticity fracture criterion for the two wire sizes



(b) Fracture shape predicted by FE simulation with ductile fracture criterion for the two wire sizes



(c) Fracture shape predicted by FE simulation with ductile fracture criterion for the two wire sizes



(d) Fracture shape predicted by FE simulation with shear fracture criterion for 12mmx5mm wire

Figure 8 : Fracture shapes predicted by FE simulations conducted with the three ductile fracture models

IV. Discussion

Except for the variations in the displacement at fracture, the simulations conducted with the shear fracture modeling parameter combinations A-D, ductile fracture modeling parameter combinations E-H, and PMP fracture modeling parameters I-L predicted the same force-displacement curves as shown in Figures 2,3 and 4. These results indicate that the predicted force-displacement responses of the wire are independent of the values of the shear, ductile and PMP fracture modeling parameters prior to the fraction initiation. The values of the shear, ductile and PMP fracture modeling parameters only affects the displacement at which fracture initiation occurs. As shown in Figures 2-4 and as presented in Figure 5, the force displacement curves predicted by the simulations conducted with the shear fracture modeling parameter combinations A-D, ductile fracture modeling parameter combinations E-H, and PMP fracture modeling parameters I-L agree with experimental curve and only differ from the experimental curve in their displacement at fracture. As shown in Figure 5, the simulations conducted with the shear parameters combination B, ductile parameters combination F and PMP parameters combination L predicted fracture initiation points that are the closest/approximately equal to the experimental fracture point. Consequently, the shear fracture modeling parameters combination B, ductile fracture modeling parameters combination F and PMP fracture modeling parameters combination L represent the appropriate or calibrated fracture modeling parameters combinations for the prediction of the tensile fracture behavior of the wire. Thus, on the basis of adjudging the ductile fracture models' appropriateness (applicability

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and reliability) by their ability to predict force-displacement curves that agree with the experimental curve up to the fracture initiation point as published by Bernauer and Brocks, (2002); Dunand and Mohr, (2010); Li et al, (2011) and Rakin et al, (2004) among others, any of the shear, ductile and porous metal plasticity failure models considered in this work can be adjudged as an appropriate fracture model for the wire. However, as shown in Figures 8(c) and (d), for the two wire sizes considered, only the simulation conducted with the shear failure model predicted the "cup and cone" fracture exhibited by the fractured experimental wire specimens shown in Figures 2(a) and (b). The simulations conducted with the PMP and the ductile failure models predicted flat and slant fracture as shown in Figures 8(a) and (b) respectively. The ability of the shear fracture model alone to predict the "cup and cone" fracture exhibited by the fractured experimental specimen indicates that out of the three fracture models considered in this work, only the shear fracture model can be adjudged as the appropriate fracture model to predict the fracture performance of the typical wire for civil engineering application considered. The inability of the ductile and porous metal plasticity fracture models to predict the flat to slant fracture propagation, which represents the fracture path or sequence associated with the cup and cone fracture behaviour exhibited by the experimental fractured wire specimens does not makes the ductile and porous metal plasticity fracture models appropriate fracture models suitable for the prediction of the fracture performance of the wire.

V. Conclusion

This study has established that it is not sufficient to choose any of the shear, the ductile or the porous metal plasticity micromechanics-based fracture models as an appropriate fracture model to predict the fracture performance of carbon steel wires for civil engineering applications on the basis of a good agreement between the experimental and FE predicted force-displacement curve alone as is generally practiced. The need to consider the capability of the micromechanics based fracture model to predict the actual fracture shape exhibited by the experimental fractured wire specimens in choosing the appropriate fracture model has been demonstrated. Out of the shear, the ductile and the porous metal plasticity ductile failure models in-built in the Abaqus finite element code considered in this work, the shear failure model has been identified as the appropriate fracture model that is able to predict the “cup and cone” fracture shape or behaviour exhibited by the fractured experimental wire specimens. Thus, FE tensile testing simulation with the phenomenological shear fracture model can thus be used to predict the fracture performance of wires for civil/structural engineering applications. This study has thus identified an appropriate ductile fracture model that is capable of predicting the fracture performance of a typical carbon steel wire for civil engineering application in terms of the wires’ force-displacement response and cup and cone fracture shape. It is hoped that, the use of FE tensile testing simulation with the phenomenological shear fracture model would be employed by engineers to predict the fracture performance of wires for civil engineering applications. This will allay the concerns on the fracture performance of the wires that are associated with the use of the traditional classical fracture mechanics approach for the prediction of the fracture performance of wires for civil engineering applications and serve as an alternative to using non-standardised traditional classical fracture mechanics specimens for the prediction of the fracture performance of wires for civil engineering reported in published literature.

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DESALINATION OF WATER USING CONVENTIONAL BASIN TYPE SOLAR STILL

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Desalination of Water using Conventional Basin Type Solar Still

S.M.Abdullah Al Faruq [α], Md.Shaheen Akter [σ], Md.Alamin [ρ] & Md.Zohrul Islam[ω]

Abstract - Solar distillation is a simple treatment of brackish (i.e. contain dissolved salts) water supplies. Distillation is the processes that can be used for water purification and can use any heating source.Solar distillation is used to yield drinking water or to yield pure water for lead acid batteries, hospitals, laboratories and in generating commercial products such as rose water.In this study, a basin type solar still (BSS) is designed, constructed and field experiments have been carried out to check the productivity of the still. The field experiments of the BSS are carried from June 07, 2011 to June 09, 2012. The maximum daily distilled water production is obtained as 3.76 lit/m2-day in July 31, 2011. Also maximum hourly distilled water production is observed as 0.46 lit/m2-hr. The average daily and hourly production rates are found as 1.80 lit/m2-day and 0.20 lit/m2-hr, respectively. The production cost of distilled water is calculated as 0.33 Tk/lit.

Keywords : solar still; solar energy; solar radiation; saline water; solar distillation.

I. Introduction

Fresh water is a basic necessity of man along with air and food. Man has been reliant on lakes, rivers and underground water reservoirs for fresh water necessities in domestic agriculture, life, and industry. However, practice of water from all sources is not always possible or desirable on account of the presence of large amount of harmful organisms and salts (Tsilingiris, 2011). Also in the south western region of Bangladesh, shallow aquifers contain arsenic, exceeding the allowable limit for Bangladesh standard (0.05 mg/l) and highly saline water exists in deep aquifers. Salinity and arsenic in water poses a serious problem for the development of appropriate water supply system. In these areas desalination techniques could be applied to meet fresh water demand produced from brackish or saline water. Most desalination techniques such as reverse osmosis, electro-dialysis, multi-stage flash etc. consume a huge amount of external energy e.g. fossil fuel/electricity (Nayak et. al., 2000).

Therefore, finding methods of using renewable energy to power the desalination process is desirable. Solar distillation is a simple desalination technique in which only solar energy is needed. A basin type solar still (BSS) is the most popular method of solar distillation compared with others due to its simplicity and longer design life. It could be one of the viable options for providing drinking water for a single house or a small community in arid, remote and coastal regions. On the other hand, uses of water from underground and surface water sources are not always desirable or possible because of the presence of large amount of salts especially in coastal areas. Excessive salinity causes various health hazard and diseases in coastal region. In these areas basin type solar still (BSS) could be a suitable technique for supplying potable water. The method will serve the community with fresh water reducing the harmful effect. Desalination or desalinization denotes to any of several processes that remove extra salt and other minerals from water. Generally, desalination can also denote to the removal of salt and other minerals from water. Solar distillation is a the simple treatment of brackish that contains dissolved salts. Solar distillation has been in practice for a long time. Tripathi and Tiwari (2005) have proposed a thermal modeling of passive and active solar stills for different depths of water by using the concept of solar fraction. In this experiment, it is observed that the internal convective heat transfer coefficient decreases with the increase of water depth in the basin due to decrease in water temperature. Tanoak and Nakatake (2006) conducted an experiment on effect of inclination of external flat plate reflector of basin type still in winter. They proposed a new geometrical method for calculating the solar radiation reflected by the inclined external reflector and then absorbed on the basin liner. Elsarrag (2008) conducted an experiment on evaporation rate of a novel tilted solar liquid desiccant regeneration system. In this study a corrugated blackened surface is used to heat the desiccant and an air flow is used to regenerate calcium chloride solution. In this study, a basin type solar still (BSS) is designed, constructed and field experiments have been carried out on the roof of the Civil Engineering building of Khulna University of Engineering & Technology (KUET) from June 07, 2011 to June 09, 2012 to check the productivity of the still.

II. Desalination Practice in Bangladesh

Due to scarcity of fresh water and presence of excessive arsenic in ground water in Bangladesh increases the demand of alternative source of drinking water. On the other hand surface water is not usable

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due to large scale of salinity. Thus solar desalination is one of the alternative options available for water supply in the arsenic affected areas. The water produced by solar desalination is completely free of salinity and can be mixed with tube well water to increase the volume of water for drinking water supply. The technology cannot produce an adequate quantity of water as a result of cost. The system required further development for use in water supply in rural areas. Gani and Shama (1993) conducted an experiment on basin type solar still from August to October. They used Ferro-cement basin with glass plate cover for storing saline water. The average daily production rate was found as 457 ml/m^2.Mahmood and Rahman (1994) studied on basin type solar still. They used Ferro-cement basin with glass plate cover for storing saline water. The highest average daily output rate was found as 487.81 ml/m^2 in the month of May. Jafor (2011) studied on basin type solar still. He designed and constructed a basin type solar still (BSS) for solar desalination purpose which has an airtight rectangular Ferro-cement basin (90cm$\times$60cm$\times$5cm). The average daily and hourly production rate was found as 2.88 lit/m^2-day and 0.233 lit/m^2-hr, respectively.

III. Methods of Desalination

a) Solar Distillation

Solar stills should normally only be considered for removal of dissolved salts from water. If there is a best between polluted surface water and brackish ground water, it will usually be inexpensive to use a slow sand filter or other treatment method. If there is no fresh water, the majorre placements are rainwater collection, desalination and transportation. Solar distillation has been used for many years for relatively small plant outputs. Although solar stills are comparatively easy to operate and build, there are few, if any, economies-of-scale linked with larger plants. For example, the largest solar stills yet tested have produced only a few thousand gallons of water per day. A well designed solar still can produce 2 to 4 liters of water per square meter of basin area. A sketch of a solar still is shown in Figure 1.

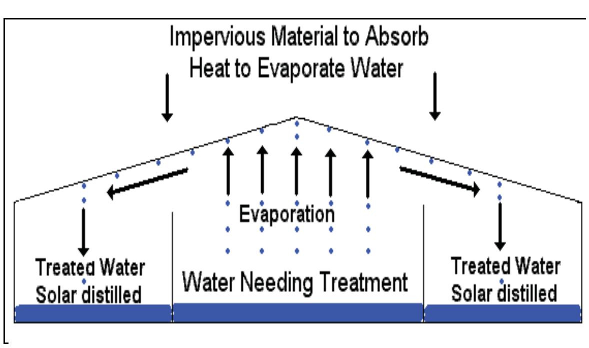


Figure 1 : Section of a typical solar still

b) The Basin Type Solar Stil

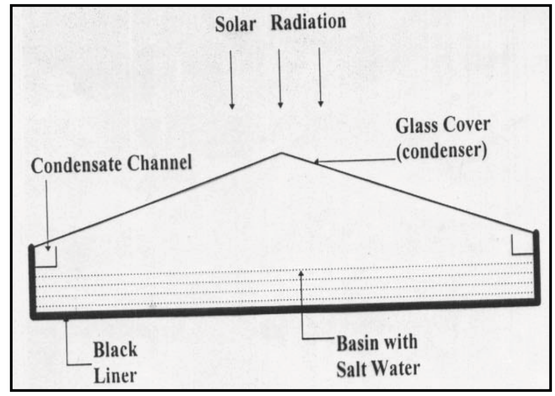
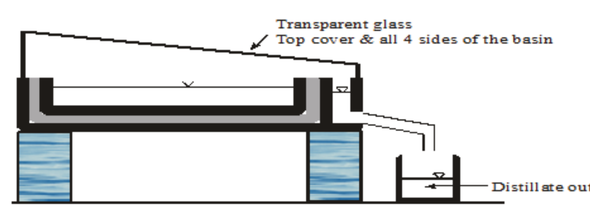


Figure 2 : Section of a typical basin type solar still

IV. Design of Basin Type Solar Still

A conventional basin type solar still (BSS) is designed and constructed in previous year using locally available materials. The still is 100cm long, 70cm wide, 10cm deep and made of 2.5cm thick Ferro-cement materials which is covered with transparent glass of thickness 5mm as condensation surface. The inclination of the glass is 10°with horizontal. The four sides of basin are covered by glass with lower and upper height is 7.5 cm and 22.5 cm, respectively. The still have rectangular basin 90cm long, 60cm wide, 5cm deep and also made of 2.5cm thick Ferro-cement materials. Inside surface and bottom of the basin are insulated by Styrofoam (known as cork sheet) of thickness 2.5cm for protection of heat transfer. To increase the productivity of the still the lower and upper heights are reduced by 2 inch in February 25, 2012. Figure 3 shows the schematic diagram of the BSS.



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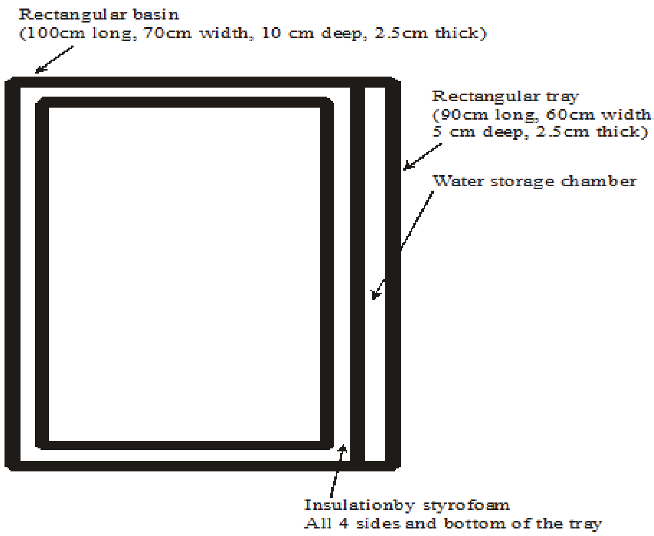

Figure 3 : Schematic diagrams of the BSS

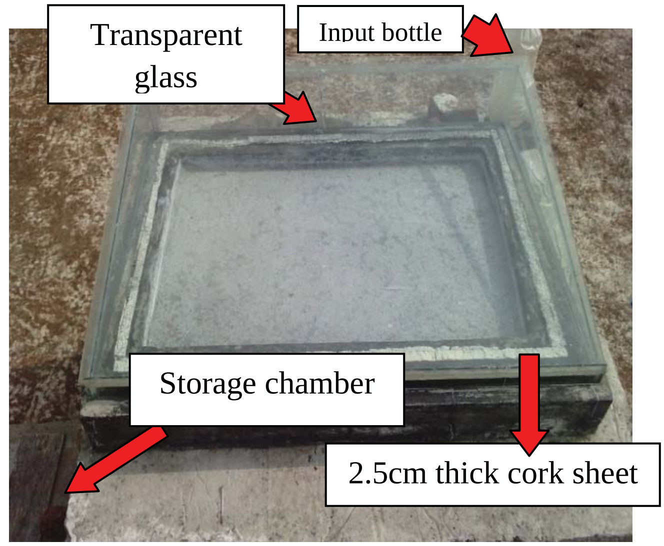

Figure 4 : Photograph of field experiment

V. Field Experiment

The field experiment on the constructed BSS was carried out on the top roof of Civil Engineering building from June 07, 2011 to June 09, 2012. The daily production of distillated water was collected into a bottle everyday approximately two hours after the sunset. Hourly production and ambient air temperature for May 20, 21, and 22 in 2012 were measured during the day time. Figure 4 shows the photograph of the field experiment.

VI. Data Collection

Daily production of distilled water was measured approximately two hours after sunset and tabulated in Table 1. Hourly production and ambient air temperature for May 20, 21, and 22 in 2012 were also measured during the day time and given in Table 2.

Table 1 : Daily distilled water production from June 07, 2011 to June 09, 2012

Day	Daily distilled water production (ml/day)									
	Year (2011)					Year (2012)				
	June	July	September	November	December	February	March	April	May	June
1	-	285	-	-	520	-	1300	1730	-	950
2	-	510	-	-	610	-	1010	1750	-	1125
3	-	464	-	-	645	-	1240	1745	-	1735
4	-	1076	-	-	580	-	1325	1695	-	1670
5	-	1180	-	-	540	-	1125	1795	-	1365
6	-	1100	1050	-	580	-	1250	1250	-	1245
7	2080	1410	980	-	530	-	1305	1710	-	1490
8	1489	975	755	-	690	-	1415	1605	-	1580
9	1927	800	390	-	710	-	1270	1000	-	1610
10	1705	1393	780	-	530	-	-	1468	-	-
11	1078	1100	1010	-	425	-	-	1415	-	-
12	602	1300	1105	-	400	-	-	1512	-	-
13	1626	1105	1073	-	150	-	1100	1455	-	-
14	901	1040	928	920	120	-	1225	1670	-	-
15	923	1180	790	450	170	-	1500	1765	-	-
16	647	1020	920	230	160	-	1350	1900	-	-
17	529	980	200	760	210	-	1330	1860	-	-
18	201	1073	830	785	280	-	1260	-	-	-
19	100	420	925	410	190	-	705	-	-	-
20	351	105	1015	500	300	-	1255	-	-	-
21	627	250	1050	710	240	-	1500	-	-	-

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22	920	110	970	615	260	-	1640	-	-	-
23	1302	1675	530	670	280	-	1630	-	1705	-
24	781	928	840	430	300	-	1660	-	1735	-
25	407	948	780	510	440	-	1640	-	1745	-
26	177	1575	1025	535	500	-	1610	-	1150	-
27	751	1608	1070	675	540	1210	1650	-	1170	-
28	400	1625	710	600	510	1225	1710	-	1250	-
29	619	1973	605	595	530	1120	1760	-	1040	-
30	867	2027	965	572	560	-	1670	-	1025	-
31	-	2105	-	-	580	-	1780	-	876	-

Table 2 : Hourly distilled water production and ambient air temperature for three typical days of May 20, 21 and 22, 2012

Time of Day	Hourly distilled water production (ml/hr)			Ambient air temperature (ºC)		
	May 20	May 21	May 22	May 20	May 21	May 22
7.00 am	0	0	0	32	32	32
8.00 am	8	10	5	33.5	32.5	33.5
9.00 am	16	30	5	33.5	33.5	34.5
10.00 am	40	34	21	36.5	30.5	35
11.00 am	120	98	46	39	32.5	35.5
12.00 pm	166	140	90	40	34	37
1.00 pm	180	256	187	38	36	37.5
2.00 pm	195	224	222	33.5	34	38.5
3.00 pm	197	223	214	33.5	33.5	38
4.00 pm	189	191	226	33	33.5	38.5
5.00 pm	144	155	167	34	33.5	38.5
6.00 pm	130	135	170	36	33	36.5
7.00 pm	120	115	106	36	32.5	36
8.00 pm	36	51	67	36.5	32	30

VII. Data Analysis

Data collection and measurement in Section 3.3 were used to calculate the daily (June 07, 2011 to June 09, 2012) and hourly (May 20, 21 and 22, 2012) distilled water production per unit surface area of the saline water in the trough for the BSS and given in Table 3 and Table 4 respectively.

Table 3 : Daily distilled water production rate from June 07, 2011 to June 09, 2012

Day	Daily distilled water production (Lit/m^2-day)									
	Year (2011)					Year (2012)				
	June	July	September	November	December	February	March	April	May	June
1	-	0.51	-	-	0.93	-	2.32	3.08	-	1.69
2	-	0.91	-	-	1.1	-	1.80	3.13	-	2.01
3	-	0.83	-	-	1.15	-	2.21	3.11	-	3.1
4	-	1.92	-	-	1.03	-	2.37	3.03	-	2.98
5	-	2.11	-	-	0.96	-	2.01	3.21	-	2.44
6	-	1.96	1.88	-	1.04	-	2.23	2.23	-	2.22
7	3.71	2.52	1.75	-	0.95	-	2.33	3.05	-	2.67
8	2.66	1.74	1.35	-	1.23	-	2.53	2.87	-	2.82
9	3.44	1.43	0.70	-	1.27	-	2.27	1.78	-	2.88
10	3.05	2.49	1.40	-	0.95	-	-	2.62	-	-

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11	1.93	1.96	1.80	-	0.76	-	-	2.53	-	-
12	1.1	2.32	1.97	-	0.71	-	-	2.7	-	-
13	2.9	1.97	1.92	-	0.27	-	1.96	2.6	-	-
14	1.61	1.86	1.66	1.64	0.21	-	2.19	2.98	-	-
15	1.65	2.11	1.41	0.80	0.30	-	2.68	3.15	-	-
16	1.16	1.83	1.64	0.41	0.26	-	2.41	3.39	-	-
17	0,96	1.75	0.36	1.36	0.38	-	2.38	3.32	-	-
18	0.36	1.92	1.48	1.4	0.50	-	2.25	-	-	-
19	0.18	0.75	1.65	0.73	0.34	-	1.26	-	-	-
20	0.63	0.19	1.81	0.90	0.54	-	2.24	-	3.1	-
21	1.12	0.45	1.88	1.27	0.43	-	2.68	-	3.19	-
22	1.64	0.20	1.73	1.1	0.46	-	2.93	-	2.99	-
23	2.33	3.00	0.95	1.20	0.5	-	2.91	-	3.04	-
24	1.4	1.66	1.5	0.77	0.54	-	2.96	-	3.1	-
25	0.73	1.70	1.4	0.91	0.785	-	2.93	-	3.12	-
26	0.32	2.81	1.83	0.96	0.89	-	2.88	-	2.05	-
27	1.34	2.87	1.91	1.21	0.96	2.16	2.95	-	2.09	-
28	0.71	2.9	1.27	1.1	0.91	2.19	3.05	-	2.23	-
29	1.1	3.52	1.1	1.1	0.95	2.00	3.14	-	1.86	-
30	1.55	3.62	1.72	1.02	1.00	-7	2.98	-	1.83	-
31	-	3.76	-	-	1.04	-	3.18		1.56	-

Table 4 : Hourly distilled water production and ambient air temperature for three typical days of May 20, 21 and 22, 2012

Time of Day	Hourly distilled water production (lit/m^2-hr)			Ambient air temperature (ºC)		
	May 20	May 21	May 22	May 20	May 21	May 22
7.00 am	0	0	0	32	32	32
8.00 am	0.014	0.018	0.009	33.5	32.5	33.5
9.00 am	0.029	0.054	0.009	33.5	33.5	34.5
10.00 am	0.071	0.061	0.038	36.5	30.5	35
11.00 am	0.214	0.175	0.082	39	32.5	35.5
12.00 pm	0.296	0.251	0.161	40	34	37
1.00 pm	0.321	0.457	0.333	38	36	37.5
2.00 pm	0.348	0.400	0.396	33.5	34	38.5
3.00 pm	0.352	0.398	0.382	33.5	33.5	38
4.00 pm	0.338	0.341	0.404	33	33.5	38.5
5.00 pm	0.257	0.277	0.298	34	33.5	38.5
6.00 pm	0.232	0.241	0.304	36	33	36.5
7.00 pm	0.214	0.205	0.189	36	32.5	36
8.00 pm	0.064	0.091	0.119	36.5	32	30

VIII. Results

Figure 5 shows the variation of the daily production rate throughout the study period. It is observed that the production is higher in summer time (April to June) and lower in winter time. The average production rate is found as 1.8 lit/m^2-day, whereas the maximum and minimum productions are found respectively as 3.76 and 0.21 lit/m^2-day for July 2011 and December 2012.

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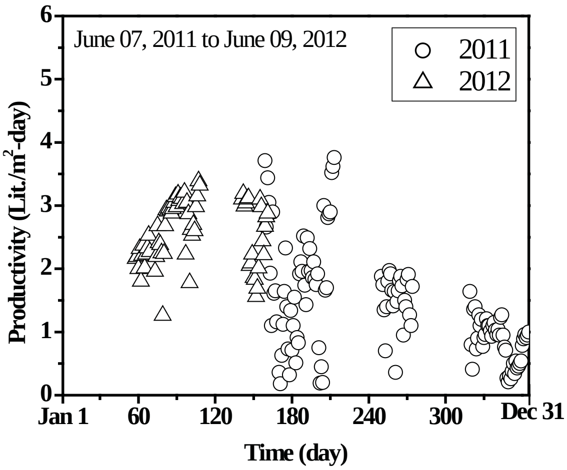

Figure 5 : Variation of production rate of the BSS

Figure 6 shows the variation of the hourly production and ambient air temperature for three typical days May 20 to 22, in 2012. The maximum air temperatures are obtained between 12:00 to 13:00 but the maximum hourly productions are observed between 14:00 to 16:00. The maximum hourly productions are found as 0.35 (15:00), 0.46 (13:00) and 0.40 (14:00) lit/m^2-hr for May 20 to 22 of 2012, respectively.

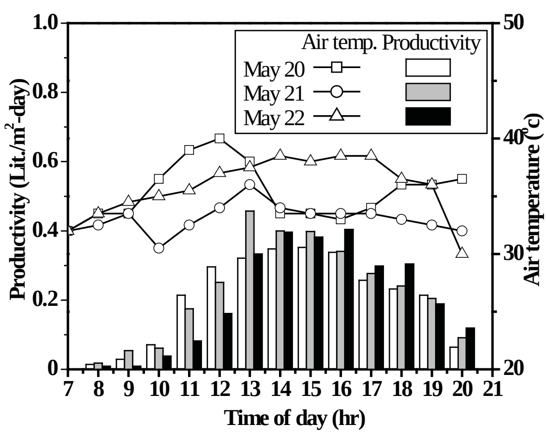

Figure 6 : Variation of hourly productivity and ambient air temperature

Figure 7 shows the average daily production for various months for the study period and given in Table 5. The highest and lowest values are found respectively as 2.87 and 0.73 lit/m^2-day for April and December.

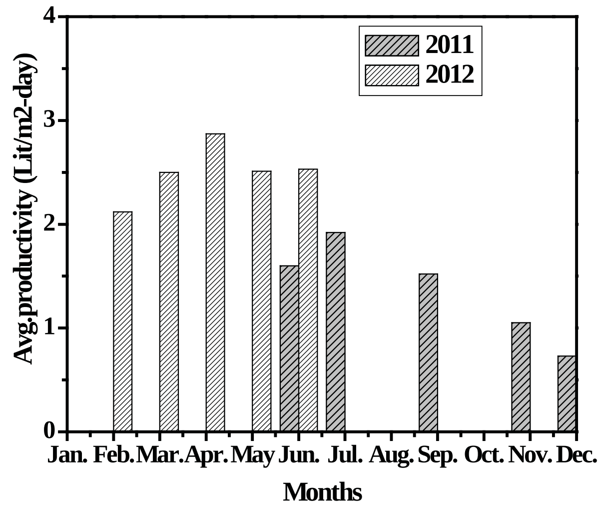

Figure 7 : Comparison of monthly production rates

Table 5 : Average daily production rate for the BSS

Name of months	Average production rate (lit/m^2-day). of conventional BSS	
	Year (2011)	Year (2012)
January	-	-
February	-	2.12
March	-	2.50
April	-	2.87
May	-	2.51
June	1.60	2.53
July	1.92	-
August	-	-
September	1.52	-
October	-	-
November	1.05	-
December	0.73	-

Figure 8shows the comparison of maximum daily production rates for the conventional basin type still with the stepped one (Imran, 2012) throughout the study period and tabulated in Table 6. It is found that the production rates are approximately 16% higher in case of stepped basin type still.

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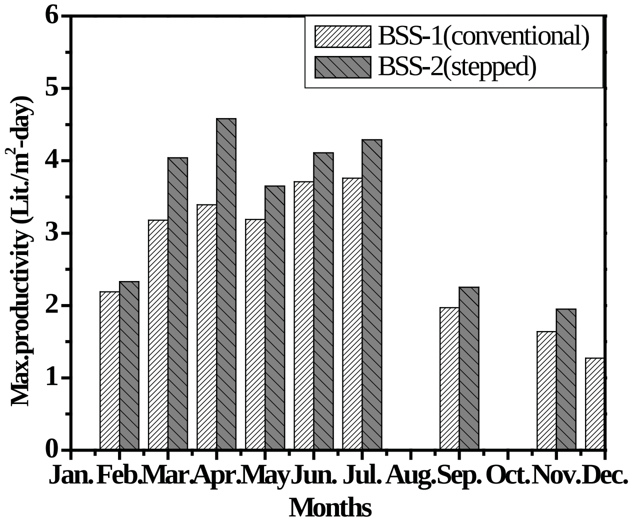

Figure 8 : Comparison of maximum production rates of different BSS

Table 6 : Comparison of maximum daily production rates (June 11, 2011 to June 9, 2012)

Name of months	Maximum daily production rate (lit/m^2-day)	
	Conventional BSS	Stepped
January	-	-
February	2.19	2.33
March	3.18	4.04
April	3.39	4.58
May	3.19	3.65
June	3.71	4.11
July	3.76	4.29
August	-	-
September	1.97	2.25
October	-	-
November	1.64	1.95
December	1.27	1.59
Total	24.3	28.79
Production rates = (28.79-24.3) = 4.49 % of production rates = (4.49 ÷ 28.79) ×100 = 15.6		

IX. Cost Analysis

Table 7 shows the cost analysis for the BSS. The construction cost of the still is estimasted as Tk. 1200 with a design life of 10 years. The production cost of the distilled water is calculated as 0.33 Tk./lit.

Table 7 : Cost analysis for the BSS

Sl. No.	Item Description	Unit	Rate (Tk.)	Quantity	Amount (Tk.)
1	Cement	kg	9	35	315
2	Sand	cft	20	2	40
3	G.I. wire	kg	75	2.5	188
4	Transparent glass	sft	45	8	360
5	Miscellaneous				297
Total =					1200
Design life of the BSS = 10 years Constrution cost of the BSS = Tk. 1200 Operation and maintenance cost = Tk. 0 The average daily production of the BSS = 1.01 lit/day Total production of water in the design life =10×365×1.01 = 3687 lit. Production cost of water = (1200 ÷ 3687) = 0.33 Tk./lit.					

Results

The scarcity of fresh water is increasing day by day in arid, remote and coastal areas. Also in the south western region of Bangladesh, shallow and deep aquifers contain arsenic and high salinity, respectively. Salinity and arsenic in water poses a serious problem for the development of appropriate water supply system. In these areas desalination techniques could be applied to meet fresh water demand produced from brackish or saline water. Among desalination techniques, solar desalination is a simple and low cost technique due to no use of fossil fuel or electricity. A basin type solar still (BSS) is the most popular method of solar distillation compared with others due to its simplicity and longer design life. In this study, a conventional basin type solar still (BSS) is designed, constructed and field experiments have been carried out fromJune 07, 2011 to June 09 2012 on the top roof of Civil Engineering building. The average daily and hourly production rates are found as 1.80 and 0.20 lit/m^2-hr, respectively. The maximum daily and hourly productions are found as 3.76 lit/m^2-day in July, 2011 and 0.46 lit/m^2-hr, respectively. It is observed that the production is higher in summer time (April to June) and lower in winter time. The average daily production for various months with highest and lowest values is found as 2.87 and 0.73 lit/m^2-day for April and December, respectively. It is found that the production rates are approximately 16% higher in case of stepped basin type still. The construction cost of the still is calculated as 1200 Tk. /BSS and the water production cost is estimated as 0.33 Tk/lit with a design life of 10 years.

X. Conclusion

The design, construction and operation of BSS are very simple. Also operational and maintenance cost is very low. Since the initial and water production cost of the still is low, it could be one of the acceptable options for providing drinking water for a single house or a small community in arid, remote and coastal regions.

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Determination of the Appropriate Plasticity Hardening Model for the Simulation of the Reverse Bending and Straightening of Wires for Civil Engineering Applications

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Keywords : bauschinger effect, combined hardening, finite element simulation, isotropic hardening, laminations, reverse bending strain reversal wires.

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DETERMINATION OF THE APPROPRIATE PLASTICITY HARDENING MODEL FOR THE SIMULATION OF THE REVERSE BENDING AND STRAIGHTENING OF WIRES FOR CIVIL ENGINEERING APPLICATIONS

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Keywords : bauschinger effect, combined hardening, finite element simulation, isotropic hardening, laminations, reverse bending, strain reversal, wires.

I. Introduction

Carbon steel wires are used in the construction of many civil engineering structures. Specifically, carbon steel wires are used as pre-stressing tendons and as suspension and/or cable-stayed bridge wires. Carbon steel wires are also incorporated into flexible pipes used for offshore oil and gas transportation as axial stress reinforcement. These wires are subject to a number of non-destructive tests to detect defects that could threaten their integrity in service. One of these tests is the reverse bending and straightening test which involves bending of the wire over the rotating left hand roller, reverse bending of the wire over the rotating middle roller and finally straightening of the wire over the rotating right hand roller as shown in Figure 1 to detect laminations in the wires. A lamination is an elongated line-type defect or a long crack that is usually invisible and usually parallel to the surface of metal products (such as wires) produced through rolling or drawing process (Smith et al, 1957). It is essential to detect laminations in wires/bars used for civil engineering applications as the catastrophic rupture of pre-stressed concrete pipes has been attributed to the presence of long straight pre-service longitudinal cracks (i.e. laminations) in the pre-stressing wires used for pre-stressing the ruptured pre-stressed concrete pipes by the United States Bureau of Reclamation, 1994.

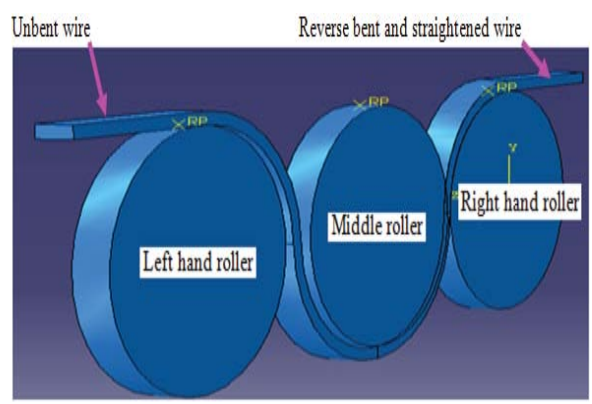

Figure 1 : Industrial reverse bending equipment with three rollers

In the current work, three dimensional FE simulations were conducted to identify the appropriate hardening model for the simulation of the wire reverse bending and straightening test. The FE simulation of the wire reverse bending and straightening test was conducted as a part of the research to investigate the effects of the combination of reverse bending and laminations on the tensile properties of a typical wire used for civil engineering applications. FE simulation was employed for the investigation of the effects of the combination of reverse bending and laminations on the tensile properties of the wire because it was not experimentally possible to simulate the long straight longitudinal cracks or laminations that are parallel to the

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length of the wire specimens (such as the lamination found in the pre-stressing wires used to pre-stress the ruptured pre-stressed concrete pipe) using machining techniques. For an accurate simulation of the wires' reverse bending and straightening test, it is essential to employ an appropriate material constitutive model, particularly the plasticity hardening model that is able to capture the wires' behavior(s) during the various stages (bending, reverse bending and straightening) of the reverse bending and straightening test/process.

Most of the published literature on reverse bending of metal products (for example Firat (2007); Gau and Kinzel (2001)), are in sheet metal forming. The importance of using appropriate material constitutive model(s) in the finite element (FE) formability and springback predictions in sheet metal forming simulations has been emphasised by Gau and Kinzel (2001), Firat, (2007) and Taherizadeh et al, (2010). Material constitutive models that have been used in FE sheet metal forming predictions and springback analyses include the isotropic, kinematic and anisotropic hardening models. Combinations of two or more of these hardening models have also been used for FE sheet metal forming simulation and spring back prediction. Firat, (2007), Zhao and Lee, (1999), and Ken-ichiro, (2001) observed that the isotropic hardening model when used in reverse bending simulation overestimates the hardening component and does not predict the through-thickness stress distribution accurately because it does not consider the Bauschinger effect. The Bauschinger effect is responsible for the reduction in both the fatigue strength and the static yield strength of a metal when it is subjected to strain reversal (Takeda and Chen (1999). Zhao and Lee, (1999) reported that "the kinematic hardening rule underestimates the hardening component and exaggerates the Bauschinger effect". The shortcomings associated with using the isotropic and kinematic hardening models has led to an emerging new standard of models with mixed or combined hardening (a combination of two or more different hardening models) which has proven to increase the numerical reliability of sheet metal formability and springback predictions (Carbonnie et al, 2008).

An acceptable material model should be able to capture the many different material phenomena that occur during plastic deformation, such as work hardening and the Bauschinger effect, and should be able to provide the best possible fit to the actual material properties (Taherizadeh et al, 2010). Consequently, it is essential to understand the applicability of these models and their limitations in order to increase the accuracy of FE reverse bending simulations. To the best of the authors' knowledge, no guidance on the appropriate plasticity hardening model for the simulation of the reverse bending and straightening of a wire which involves "double" strain reversal (strain reversal due to reverse bending and strain reversal due to straightening of the wires) has been published.

In this work, the three dimensional FE simulations were conducted using the isotropic elastic-plastic hardening model and the combined hardening plasticity models in-built in the Abaqus v 6.9.3 FE software materials library. The FE simulations with the isotropic and the combined hardening models were conducted in combination with the phenomenological shear damage and failure criterion. The details of the phenomenological shear failure model can be found in the work of Adewole and Bull, (2013). The details of the isotropic elastic-plastic and isotropic elastic-combined hardening plasticity models are presented in the following sections.

a) Isotropic Elastic-Plasticity Model

The isotropic elastic-plasticity model in Abaqus is based on a linear isotropic elasticity theory and a uniaxial-stress, plastic-strain strain-rate relationship (Simulia, 2007). The elastic aspect of the model is defined in terms of its volumetric and deviatoric components given in equations (1) and (2) respectively obtained from Simulia, (2007). The model is based on a von Mises yield surface with the yield function, f , given in equation (3) and a flow rule given in equation (4) obtained from (Simulia, 2007).

$$p = -K\varepsilon_{vol} \tag{1}$$

$$S = 2Ge^{el} \tag{2}$$

$$f = q = \sqrt{\frac{3}{2}S : S} \tag{3}$$

$$de^{pl} = d\bar{e}^{pl} n \tag{4}$$

Here p is the hydrostatic stress, ε_{vol} is the volume strain, S is the deviatoric stress, e^{el} is the deviatoric elastic strain, q is the von Mises equivalent stress, e^{pl} is the deviatoric plastic strain, $\bar{e}^{pl}$ is the equivalent plastic strain, $n = \frac{3}{2}\frac{S}{q}$, K is the bulk modulus and G is the shear modulus. K and G are calculated from the Young's modulus, E , and Poisson's ratio, ν, of the material.

b) Isotropic Elastic-Combined Hardening Plasticity Model

The combined hardening model is a combination of the nonlinear kinematic and isotropic hardening models. The isotropic cyclic hardening component is based on the exponential law given in equation (5) obtained from Simulia, (2007). The kinematic hardening component is based on the

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evolution of the backstress (a nonlinear evolution of the centre of the yield surface) $\dot{\alpha}$ given in equation (6) obtained from Simulia, (2007).

$$\sigma^0 = Y_i + Q_\infty (1 - e^{-b\varepsilon pl}) \quad (5)$$

$$\dot{\alpha} = C \frac{1}{\sigma^0}(\sigma - \phi)\bar{e}^{pl} - \gamma\phi\bar{e}^{pl} \quad (6)$$

Here σ^0 is the size of the yield surface (size of the elastic range), Q_∞ (Q infinity) is the maximum increase in the elastic range, b is a material parameter that defines the rate at which the maximum size is reached as plastic straining develops and Y_i is the initial yield stress. C and γ are kinematic hardening parameters, which are material parameters that define the initial hardening modulus and the rate at which the hardening modulus decreases with increasing plastic strain, respectively (Simulia, 2007).

II. Experimental and FE Analysis Procedures

The details of the experimental and FE simulations are presented in this section.

a) Laboratory Reverse Bending, Straightening and Tensile Testing of Wires

A length of the wire was bent and reverse bent round a 100mm diameter cylindrical steel block as shown in Figure 2 and the reverse bent wire was finally straightened and cut into tensile test specimens hereinafter referred to as the experimental reverse bent and straightened (ERBS) wire specimens. The ERBS specimens and tensile specimens cut from the as-received unbent wire specimen hereinafter referred to as unbent wire tensile specimens were tested using an Instron universal testing machine (IX 4505) fitted with an Instron 2518 series load cell with a maximum static capacity of ±100 kN. The displacement was measured with an Instron 2630-112 clip-on strain gauge extensometer with a 50 mm gauge length.

Figure 2 : Experimental simulation of reverse bending of wires for civil engineering applications

b) Reverse Bending, Straightening and Tensile Testing Simulation Procedures

Figure 3 shows the model of a 305m long wire length, the left and right rollers and a guide plate introduced to prevent roller 2 from lifting vertically upward during the bending simulation. The whole model was meshed with C3D8R elements (8-node hexahedral linear brick reduced integration elements with hourglass control). The rollers and the guide plate were meshed with 3mmx3mmx3mm elements. The outer sections of the wire length were meshed with 3mmx3mmx0.5mm elements and the middle 50mm length designated as the FE tensile test specimen was meshed with a finer mesh with 3mmx1mmx0.5mm dimensions. The 3mmx1mmx0.5mm element size was established through mesh convergence studies to be the optimum mesh size required for accurate predictions of the bending and tensile behaviours of the FE tensile test specimen. The 3mm, 1mm and the 0.5mm dimensions are in the FE tensile test specimen's width, length and thickness respectively.

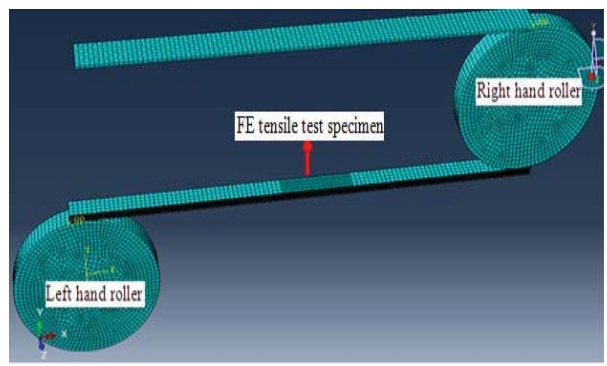

Figure 3 : Meshed model of rollers, guide plate and wire

The left hand roller was rotated in an anticlockwise direction to bend the wire round the left hand roller. After the bending simulation, the left hand roller and the right hand roller were simultaneously rotated in a clock wise and antilock wise directions respectively to unwind the wire from the left hand roller and reverse bend the wire round the right hand roller. After the reverse bending simulation, the right hand roller was rotated in a clockwise direction to unwind the reverse bent wire whilst simultaneously pulling the left hand roller longitudinally and vertically upward until the FE reverse bent test specimen was straightened. The FE reverse bent and straightened wire tensile specimen hereinafter referred to as FERBS was subjected to tensile testing by fixing the left hand end roller; the remaining section of the wire length at the left hand end of the specimen and the left hand end of the FERBS tensile specimen, and pulling the right hand end roller; the remaining section of the wire length at the right hand end of the specimen and the right hand end of the

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FERBS tensile specimen. The right hand end roller; the remaining section of the wire length at the right hand end of the specimen and the right hand end of the FERBS tensile specimen were free to move only in the tensile load direction (i.e. longitudinally in X-axis direction).

The same model arrangement shown in Figure 3 and the same boundary conditions employed for the simulations of the tensile testing of the FERBS wire specimen were employed for the simulations of the tensile testing of the unbent wire except that the bending, reverse bending and straightening simulations steps were suppressed (i.e. not conducted) during the simulations of the tensile testing of the unbent wire.

c) FE Analysis with isotropic and combined hardening plasticity models

The simulations of the tensile testing of the unbent and FERBS 5x12mm cross section steel wires were conducted with the isotropic and with the combined hardening plasticity models. Both the simulations with the isotropic and with the combined hardening plasticity models were conducted in combination with the phenomenological shears failure model inbuilt in Abaqus. The calibrated shear damage and failure modelling parameters employed for the FE simulations are fracture strain of 0.3451, shear stress ratio of 12.5, strain rate of 0.000125s^{-1} and a material parameter K_s of 0.3 which were obtained through a phenomenological curve fitting process. Details of the phenomenological curve fitting process employed to obtain the calibrated phenomenological shear failure modelling parameters for the wire considered in this work have been published by Adewole and Bull, (2013).

The material input parameters for the simulation conducted with the isotropic hardening model are the true stress and true strain values obtained from experimental tensile testing of the wires. The true stress and strain values are not presented in this paper for confidentiality purposes (i.e. due to a non-disclosure agreement on the tensile properties of the wires). Other parameters employed for the simulation conducted with the isotropic elastic–plastic model are the density of 7.6 x 10^6 kg/mm^3, Poisson's ratio of 0.3 and Young's modulus of 200 x 10^3.

The material input parameters for the simulation conducted with the combined hardening model are the initial stress Y_i at zero plastic strain (not presented for confidentiality purposes) and the calibrated combined hardening plasticity modelling parameters: kinematic hardening parameter C of 15300, gamma (γ) of 275, Q_∞ of 12000 and hardening parameter b of 0.04. The calibrated combined hardening plasticity modelling parameters were obtained through a phenomenological curve fitting process and they represent the values of the modelling parameters at which the FE predicted force-displacement curve agreed with the experimental curve. The phenomenological curve fitting was conducted by carrying out FE simulations of the tensile testing of unbent wire specimens with varying combined hardening modelling parameters until the FE predicted force-displacement curve agreed with the experimental curve up to the fracture initiation point.

III. Results

The deformed shapes of the wire showing the longitudinal axial stress (S_{11}) distribution and the through-thickness longitudinal axial stress and equivalent plastic strain profiles in the deformed wire specimen at the various stages of the reverse bending and straightening test simulation are presented in this section. In the S_{11}contour plot, positive axial stresses represent tensile axial stresses and the negative axial stresses represent compressive axial stresses. The deepest red colour at the top of the contour plot represents the highest tensile stress while the deepest blue at the bottom of the contour plot represents the highest compressive stress. For both the through-thickness longitudinal axial stress and the through-thickness equivalent plastic strain profiles, positive and negative stresses and strains represent tensile and compressive stresses and strains respectively. The stress and strain in the upper half thickness and the lower half thickness of the 5mm thick wire are plotted with 0 to 2.5mm and 0 to -2.5mm Y-axis coordinates respectively.

Throughout the bending, reverse bending, straightening and tensile testing simulations, the deformed shapes predicted by the simulations conducted with the isotropic hardening and with the combined hardening models are exactly the same. Consequently, only the deformed shapes predicted by the simulation conducted with the combined hardening models after the bending simulation, during the reverse bending simulation, after the reverse bending simulation, after the straightening simulation and after the tensile testing simulation are presented in Figures 4, 5, 6, 7 and 8 respectively. The through-thickness longitudinal axial stress and equivalent plastic strain profiles in the bent, reverse bent, and reverse bent and straightened wire specimens predicted by the simulations with the two hardening models are also presented in Figures 4, 6 and 7 respectively. The fractured experimental ERBS wire specimen is shown in Figure 8(c). The experimental force-displacement curves for the unbent and ERBS wire specimens with the force-displacement curves obtained from the simulations of the tensile testing of the unbent and FERBS wire specimens conducted with the isotropic and the combined hardening plasticity models are shown in Figures 9 and 10 respectively.

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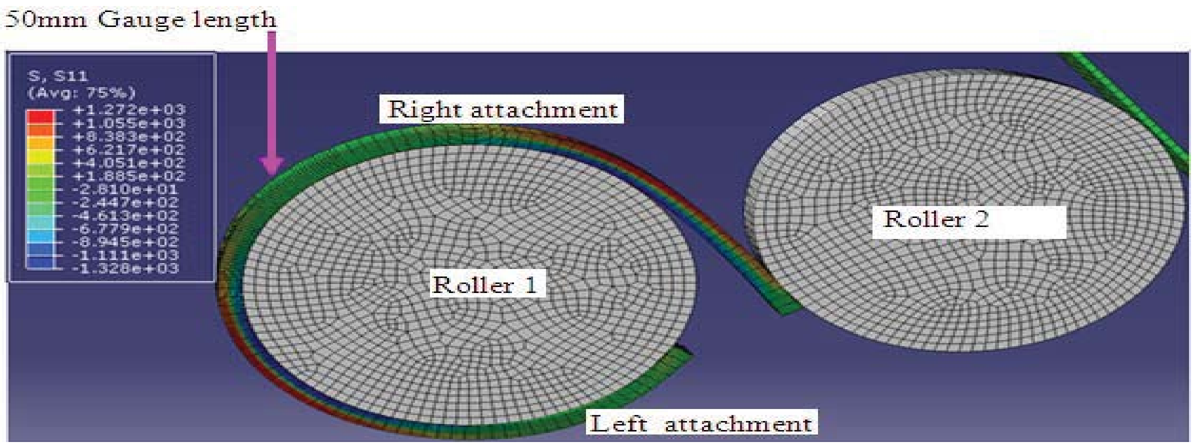
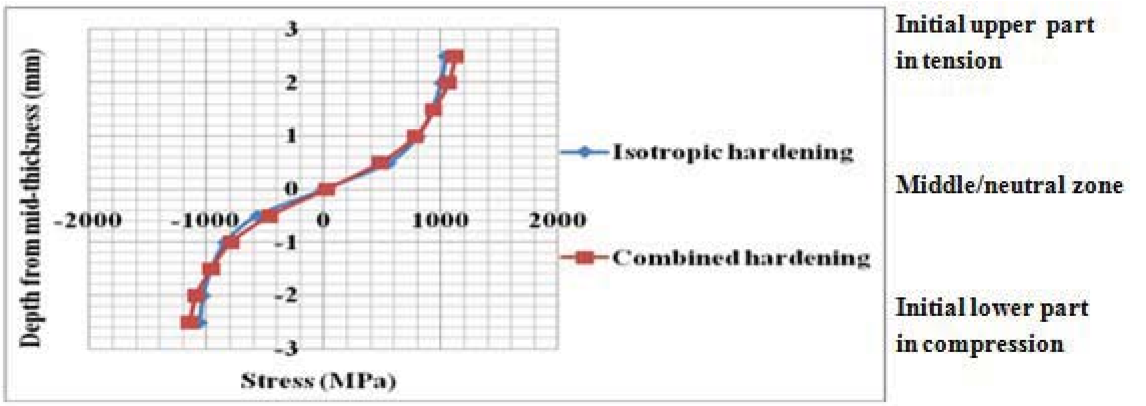
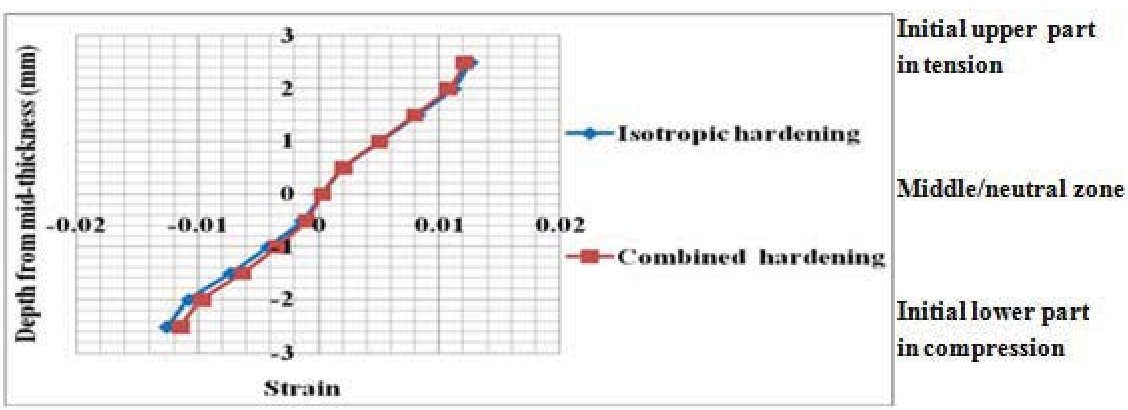


(a) Deformed shape showing longitudinal axial stress distribution



(b) Through-thickness longitudinal axial stress profile



(c) Through-thickness longitudinal axial plastic strain profile

Figure 4 : Longitudinal axial stress and equivalent plastic strain distributions and through-thickness profiles in wire after bending process simulation

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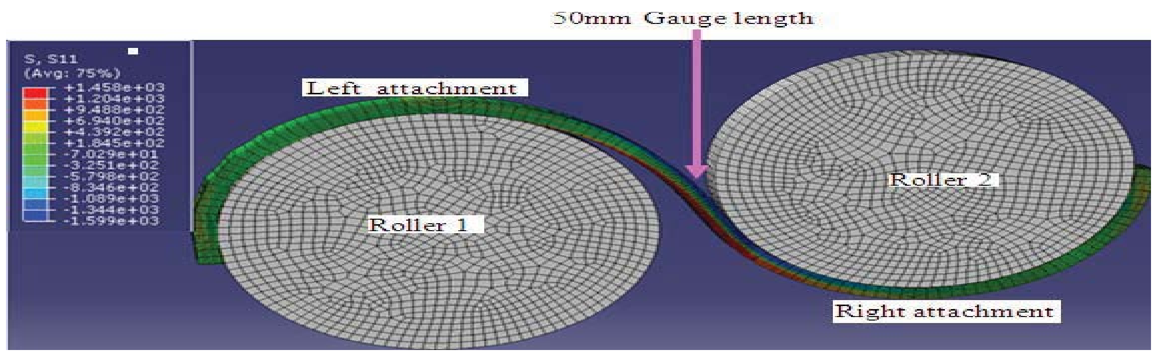
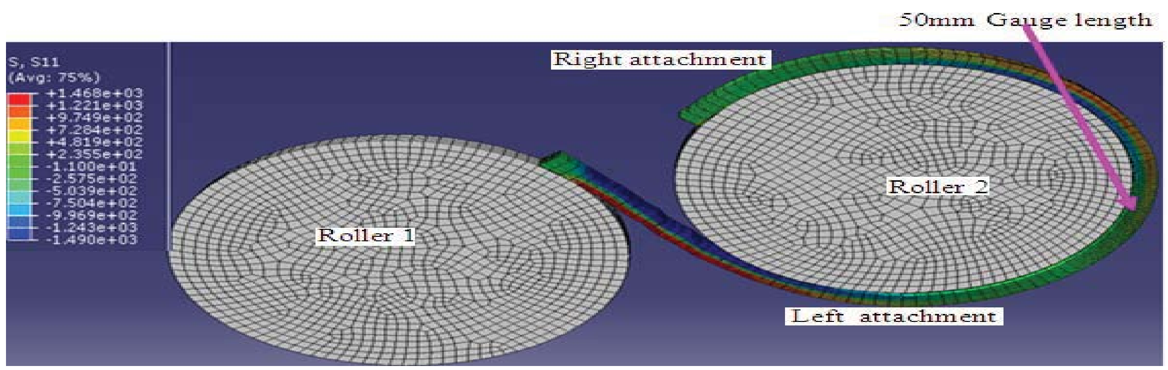
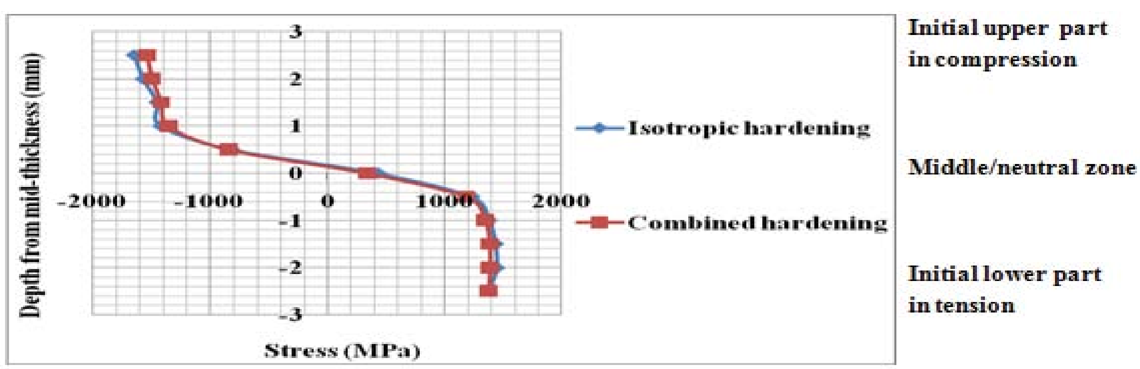


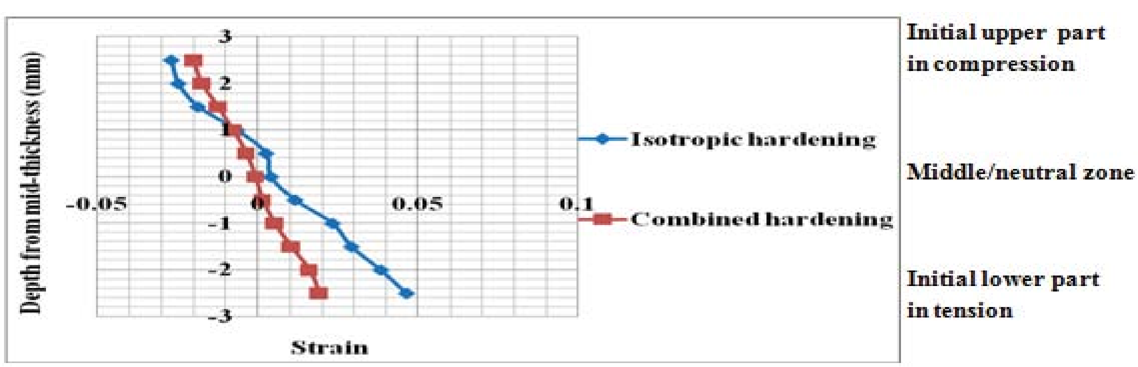
Figure 5 : Deformed shape showing longitudinal axial stress distribution in specimen during reverse bending process simulation



(a) Longitudinal axial stress distribution in whole wire length



(b) Through-thickness axial stress profile



(c) Through-thickness axial plastic strain profile

Figure 6 : Longitudinal axial stress and equivalent plastic strain distributions and through-thickness profiles in wire after reverse bending process simulation

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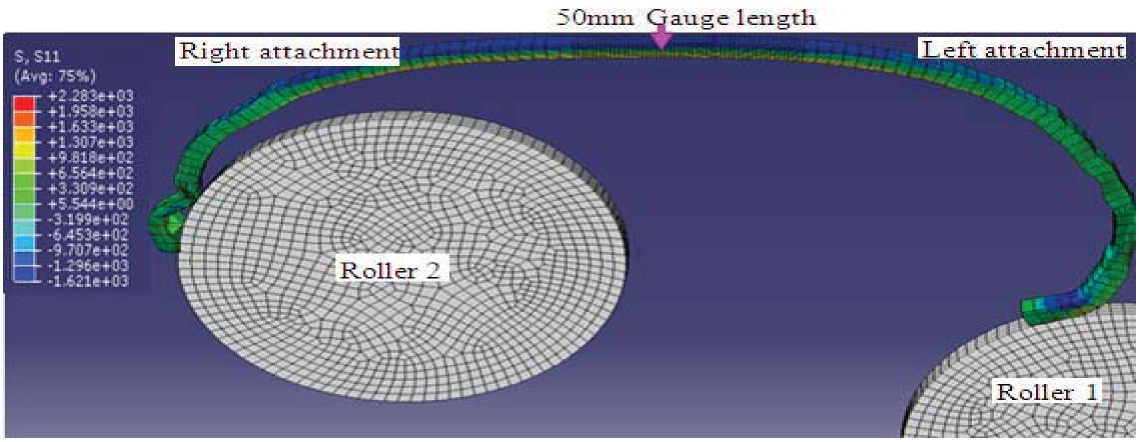
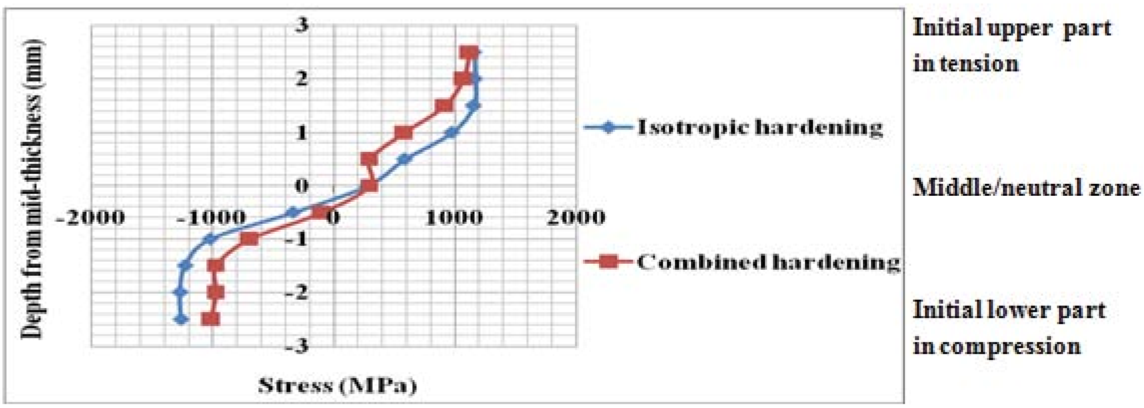
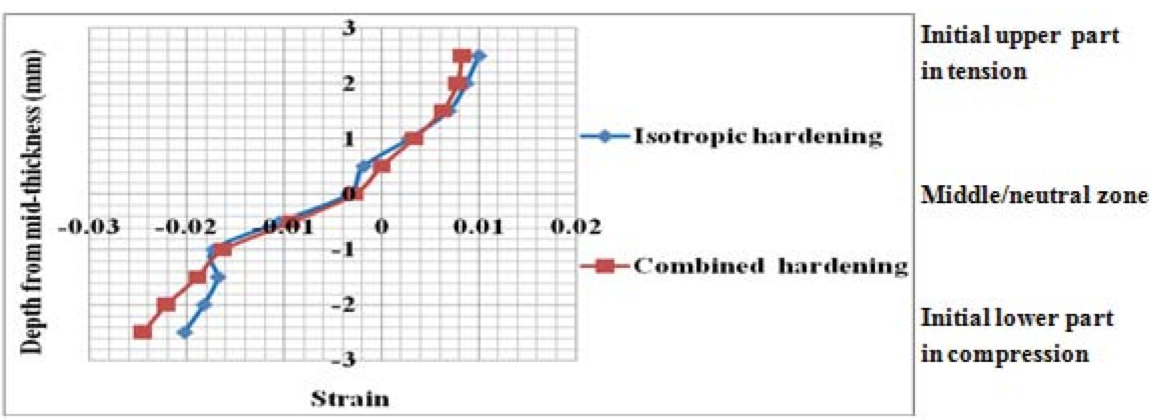


(a) Deformed shape and stress distribution in whole wire length



(b) Through thickness longitudinal axial stress profile



(c) Through-thickness axial plastic strain profile

Figure 7 : Longitudinal axial stress and equivalent plastic strain distributions and through-thickness profiles in wire after straightening process simulation

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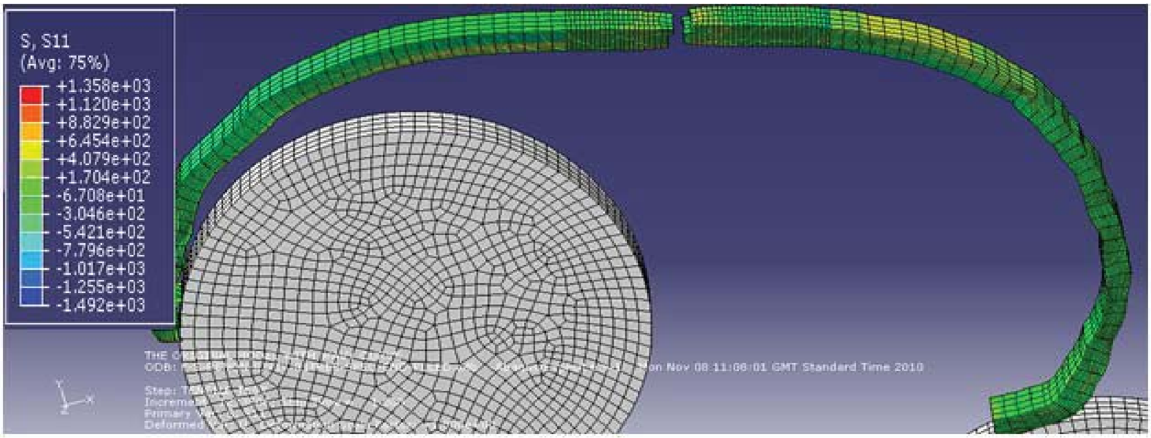
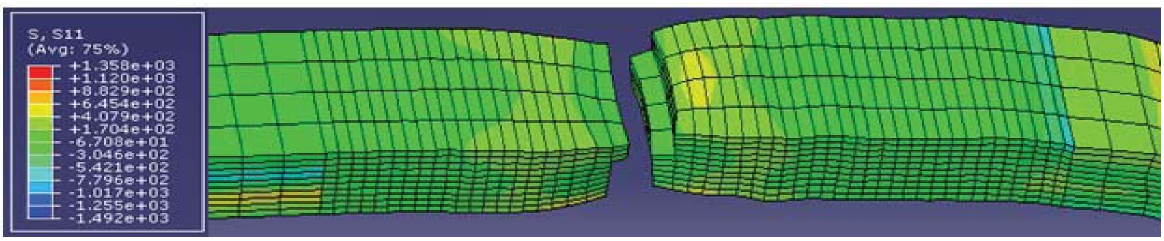
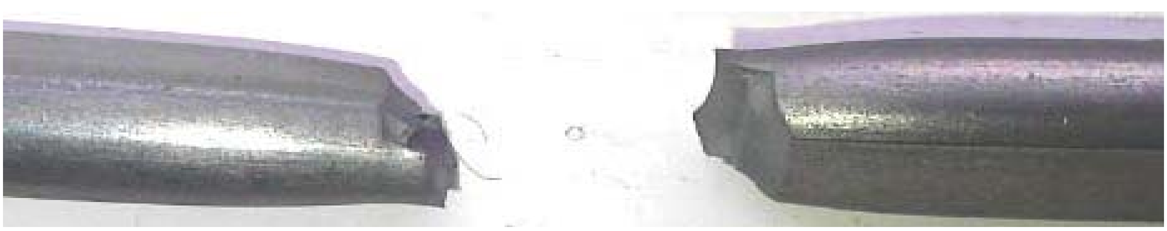


(a) Deformed shape of the whole wire length showing the Fractured FERBS specimen at the center of the whole wire length



(b) Fractured numerically RBS specimen alone



(c) Fractured experimentally RBS specimen

Figure 8 : Fractured numerically and experimentally RBS specimens after tensile testing

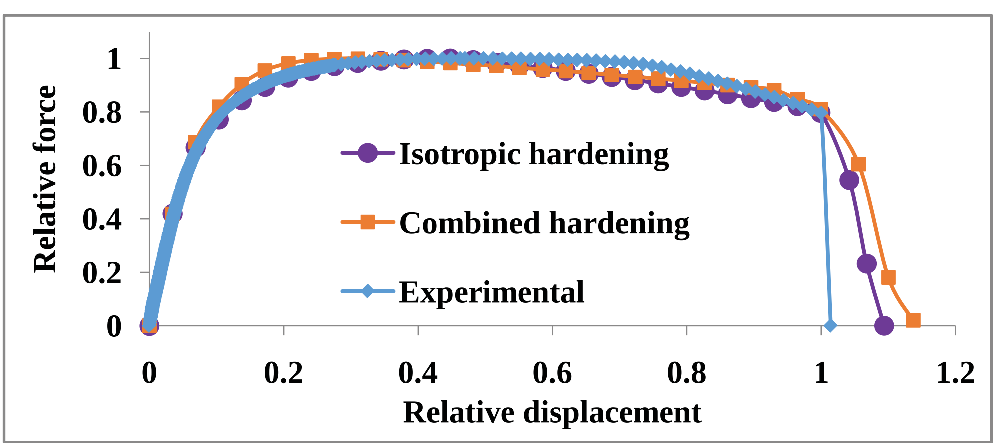


Figure 9 : Experimental and FE force-displacement curves for unbent wire specimens with isotropic and combined hardening models

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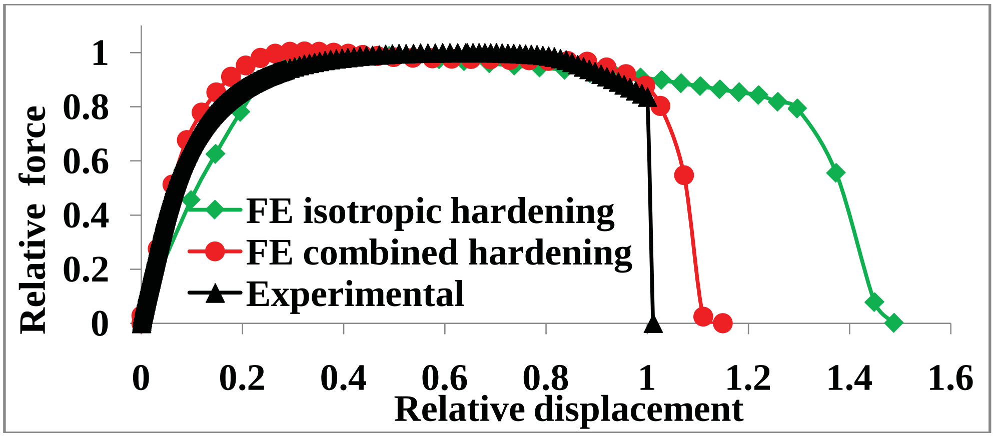

Figure 10 : Experimental and FE force-displacement curves for RBS wire specimens with isotropic and combined hardening models

XI. Discussion

As shown in Figures 4(b) and (c), there is no significant difference in the through-thickness longitudinal axial stresses and strains profiles predicted by the bending simulations conducted with both the isotropic and the combined hardening models. The simulations conducted with both hardening models predicted tensile and compressive stresses and strains at the initial upper and the initial lower parts of the wire as expected based on the deformed shape of the wire specimen shown in Figure 4(a). Also the strain profiles predicted by the simulations conducted with the two hardening models are linear as expected of a bending induced straining. This result indicates that both the isotropic and the combined hardening models are able to predict the stress and strain distributions/profiles in wires subjected to bending round the roller accurately.

Similarly, there is no significant difference in the through-thickness longitudinal axial stress profiles predicted by the reverse bending simulations conducted with both the isotropic and the combined hardening models as shown in Figures 6(b). Also the simulations conducted with the two hardening models predicted the expected strain reversal in the reverse bent wire specimen as the initial upper and initial lower parts of the wire that were subjected to tensile and compressive stresses after the bending simulation are now subjected to compressive and tensile stresses and strains respectively after the reverse bending simulation. However, the strain profile predicted by the simulation conducted with the combined hardening model is linear as expected for a bending induced straining, whereas the strain profile predicted by the simulation with the isotropic hardening model is not linear. Now, a difference between the capability/suitability of the isotropic hardening model and the combined hardening model in predicting the appropriate/expected linear strain profile in the reverse bent wire specimen that has undergone strain reversal is observed.

The straightening of the wire caused a second stress and strain reversal and induced tensile and compressive stresses and strains in the initial upper and initial lower parts of the wire specimens respectively as shown in Figure 7(a). Consequently, the initial upper part of the wire specimen has undergone a “double” stress/strain reversal resulting from the tensile stress/strain in the upper part of the wire due to bending, followed by compressive stress/strain in the upper part of the wire due to the reverse bending and followed by the tensile stress in the upper part of the wire due to straightening of the wire. Similarly the initial lower part of the wire has undergone a double strain reversal (compressive stress due to bending, tensile stress due to reverse bending and compressive stress due to straightening). There is a significant difference in the through-thickness longitudinal axial stress profiles predicted by the simulations conducted with the isotropic model and with those conducted with the combined hardening model as shown in Figure 7(b), with the simulation conducted with the isotropic hardening model predicting higher tensile and compressive stresses at the upper and the lower parts of the wire respectively. The strain profiles predicted by the two hardening models shown in Figure 5(c) are no longer linear due to the plastic straining involved in the straightening process.

The simulations of the tensile testing of the FERBS specimen conducted with the two hardening models predicted the same fracture shape shown in Figure 8(b), which agrees well with the fracture shape exhibited by the experimentally RBS specimen shown in Figure 8(c). This result indicates that the fracture process and the predicted fracture shape are largely independent of the hardening model and solely dependent on the phenomenological shear fracture model employed for the simulations with the two hardening models.

Figure 9 indicates that the force-displacement curve predicted by the simulations of the tensile testing of the unbent wire with both isotropic and combined

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hardening models agree well with the experimental curve up to the fracture initiation point. The force-displacement curve predicted by the simulation of the tensile testing of the FERBS wire with the combined hardening model agrees very well with the experimental curve throughout the elastic region, and fairly well in the plastic and fracture regions (Figure 10). However, the force-displacement curve predicted by the simulation of the tensile testing of the FERBS with isotropic hardening does not show a good agreement with the experimental curve throughout the elastic region.

It is considered that the difference between prediction and experiment is due to the fact that the isotropic hardening model does not capture the softening of the wire due to the Bauschinger effect and merely predicts that the wire has been continuously work hardened during the bending, reverse bending, and straightening operations, which is not evident in the experimental curve. The inability of the isotropic hardening model to capture the softening of the wire due to the Bauschinger effect associated with the double strain reversal involved during the reverse bending and straightening simulation also explains why the simulation conducted with the isotropic hardening model predicted higher through-thickness stress values as shown in the stress profile of the FERBS wire in Figure 7(b). Conversely, the combined hardening model adequately captured both the work hardening and Bauschinger effect that are associated with the double strain reversal involved in the reverse bending and straightening processes. Figure 10 also indicates that the isotropic hardening model predicts a displacement at fracture far higher than the experimental results. Consequently, it is concluded that the combined hardening model is the appropriate material constitutive (plasticity hardening) model for the simulation of bending, reverse bending, and straightening of carbon steel wires used for civil engineering applications.

XII. Conclusion

In this paper, it is demonstrated that both the isotropic and the combined hardening models are able to predict the bending behaviour of wires for civil engineering applications as they both predicted the stress and strain distributions and profiles in bent wire accurately. However, the isotropic hardening model does not give good predictions of the stress and strain distributions and profiles, and also does not give a good prediction of the tensile behaviour of the reverse bent and straightened wire which has undergone "double strain reversal" due to the reverse bending and straightening operations. This is due to the fact that the simulation conducted with the isotropic hardening model predicted a continuous work hardening of the wire during the bending, reverse bending, and straightening operations and did not capture the softening of the wire due to the Bauschinger effect that is associated with the double strain reversal experienced by the reverse bent and straightened wire. Conversely, the simulation conducted with the combined hardening model adequately captured both the work hardening due to bending and the softening of the wire due to the Bauschinger effect resulting from the "double" strain reversal experienced by the wire during reverse bending and straightening operations. Consequently, it is concluded that the combined hardening model serves as the most appropriate plasticity hardening model for the prediction of the behaviour of carbon steel wires subjected to bending, reverse bending, and straightening.

This work thus identifies the combined hardening model as the appropriate plasticity hardening model for wires for civil engineering applications reverse bending and straightening test simulation. This work thus provides the appropriate material constitutive model required for the numerical simulation of wires reverse bending and straightening test which is required for the numerical investigation of the effects of the combination of reverse bending and laminations on the tensile properties of a typical wire used for civil engineering applications which cannot be done experimentally as it is impossible to machine the long straight longitudinal laminations into the wires.

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8. Use the Internet for help: An excellent start for your paper can be by using the Google. It is an excellent search engine, where you can have your doubts resolved. You may also read some answers for the frequent question how to write my research paper or find model research paper. From the internet library you can download books. If you have all required books make important reading selecting and analyzing the specified information. Then put together research paper sketch out.

9. Use and get big pictures: Always use encyclopedias, Wikipedia to get pictures so that you can go into the depth.

10. Bookmarks are useful: When you read any book or magazine, you generally use bookmarks, right! It is a good habit, which helps to not to lose your continuity. You should always use bookmarks while searching on Internet also, which will make your search easier.

11. Revise what you wrote: When you write anything, always read it, summarize it and then finalize it.

12. Make all efforts: Make all efforts to mention what you are going to write in your paper. That means always have a good start. Try to mention everything in introduction, that what is the need of a particular research paper. Polish your work by good skill of writing and always give an evaluator, what he wants.

13. Have backups: When you are going to do any important thing like making research paper, you should always have backup copies of it either in your computer or in paper. This will help you to not to lose any of your important.

14. Produce good diagrams of your own: Always try to include good charts or diagrams in your paper to improve quality. Using several and unnecessary diagrams will degrade the quality of your paper by creating "hotchpotch." So always, try to make and include those diagrams, which are made by your own to improve readability and understandability of your paper.

15. Use of direct quotes: When you do research relevant to literature, history or current affairs then use of quotes become essential but if study is relevant to science then use of quotes is not preferable.

16. Use proper verb tense: Use proper verb tenses in your paper. Use past tense, to present those events that happened. Use present tense to indicate events that are going on. Use future tense to indicate future happening events. Use of improper and wrong tenses will confuse the evaluator. Avoid the sentences that are incomplete.

17. Never use online paper: If you are getting any paper on Internet, then never use it as your research paper because it might be possible that evaluator has already seen it or maybe it is outdated version.

18. Pick a good study spot: To do your research studies always try to pick a spot, which is quiet. Every spot is not for studies. Spot that suits you choose it and proceed further.

19. Know what you know: Always try to know, what you know by making objectives. Else, you will be confused and cannot achieve your target.

20. Use good quality grammar: Always use a good quality grammar and use words that will throw positive impact on evaluator. Use of good quality grammar does not mean to use tough words, that for each word the evaluator has to go through dictionary. Do not start sentence with a conjunction. Do not fragment sentences. Eliminate one-word sentences. Ignore passive voice. Do not ever use a big word when a diminutive one would suffice. Verbs have to be in agreement with their subjects. Prepositions are not expressions to finish sentences with. It is incorrect to ever divide an infinitive. Avoid clichés like the disease. Also, always shun irritating alliteration. Use language that is simple and straight forward. put together a neat summary.

21. Arrangement of information: Each section of the main body should start with an opening sentence and there should be a changeover at the end of the section. Give only valid and powerful arguments to your topic. You may also maintain your arguments with records.

22. Never start in last minute: Always start at right time and give enough time to research work. Leaving everything to the last minute will degrade your paper and spoil your work.

23. Multitasking in research is not good: Doing several things at the same time proves bad habit in case of research activity. Research is an area, where everything has a particular time slot. Divide your research work in parts and do particular part in particular time slot.

24. Never copy others' work: Never copy others' work and give it your name because if evaluator has seen it anywhere you will be in trouble.

25. Take proper rest and food: No matter how many hours you spend for your research activity, if you are not taking care of your health then all your efforts will be in vain. For a quality research, study is must, and this can be done by taking proper rest and food.

26. Go for seminars: Attend seminars if the topic is relevant to your research area. Utilize all your resources.

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27. Refresh your mind after intervals: Try to give rest to your mind by listening to soft music or by sleeping in intervals. This will also improve your memory.

28. Make colleagues: Always try to make colleagues. No matter how sharper or intelligent you are, if you make colleagues you can have several ideas, which will be helpful for your research.

29. Think technically: Always think technically. If anything happens, then search its reasons, its benefits, and demerits.

30. Think and then print: When you will go to print your paper, notice that tables are not be split, headings are not detached from their descriptions, and page sequence is maintained.

31. Adding unnecessary information: Do not add unnecessary information, like, I have used MS Excel to draw graph. Do not add irrelevant and inappropriate material. These all will create superfluous. Foreign terminology and phrases are not apropos. One should NEVER take a broad view. Analogy in script is like feathers on a snake. Not at all use a large word when a very small one would be sufficient. Use words properly, regardless of how others use them. Remove quotations. Puns are for kids, not grunt readers. Amplification is a billion times of inferior quality than sarcasm.

32. Never oversimplify everything: To add material in your research paper, never go for oversimplification. This will definitely irritate the evaluator. Be more or less specific. Also too, by no means, ever use rhythmic redundancies. Contractions aren't essential and shouldn't be there used. Comparisons are as terrible as clichés. Give up ampersands and abbreviations, and so on. Remove commas, that are, not necessary. Parenthetical words however should be together with this in commas. Understatement is all the time the complete best way to put onward earth-shaking thoughts. Give a detailed literary review.

33. Report concluded results: Use concluded results. From raw data, filter the results and then conclude your studies based on measurements and observations taken. Significant figures and appropriate number of decimal places should be used. Parenthetical remarks are prohibitive. Proofread carefully at final stage. In the end give outline to your arguments. Spot out perspectives of further study of this subject. Justify your conclusion by at the bottom of them with sufficient justifications and examples.

34. After conclusion: Once you have concluded your research, the next most important step is to present your findings. Presentation is extremely important as it is the definite medium though which your research is going to be in print to the rest of the crowd. Care should be taken to categorize your thoughts well and present them in a logical and neat manner. A good quality research paper format is essential because it serves to highlight your research paper and bring to light all necessary aspects in your research.

Informal Guidelines of Research Paper Writing

Key points to remember:

- Submit all work in its final form.
- Write your paper in the form, which is presented in the guidelines using the template.
- Please note the criterion for grading the final paper by peer-reviewers.

Final Points:

A purpose of organizing a research paper is to let people to interpret your effort selectively. The journal requires the following sections, submitted in the order listed, each section to start on a new page.

The introduction will be compiled from reference matter and will reflect the design processes or outline of basis that direct you to make study. As you will carry out the process of study, the method and process section will be constructed as like that. The result segment will show related statistics in nearly sequential order and will direct the reviewers next to the similar intellectual paths throughout the data that you took to carry out your study. The discussion section will provide understanding of the data and projections as to the implication of the results. The use of good quality references all through the paper will give the effort trustworthiness by representing an alertness of prior workings.

Writing a research paper is not an easy job no matter how trouble-free the actual research or concept. Practice, excellent preparation, and controlled record keeping are the only means to make straightforward the progression.

General style:

Specific editorial column necessities for compliance of a manuscript will always take over from directions in these general guidelines.

To make a paper clear

· Adhere to recommended page limits

Mistakes to evade

- Insertion a title at the foot of a page with the subsequent text on the next page
- Separating a table/chart or figure - impound each figure/table to a single page
- Submitting a manuscript with pages out of sequence

In every sections of your document

· Use standard writing style including articles ("a", "the," etc.)

· Keep on paying attention on the research topic of the paper

· Use paragraphs to split each significant point (excluding for the abstract)

· Align the primary line of each section

· Present your points in sound order

· Use present tense to report well accepted

· Use past tense to describe specific results

· Shun familiar wording, don't address the reviewer directly, and don't use slang, slang language, or superlatives

· Shun use of extra pictures - include only those figures essential to presenting results

Title Page:

Choose a revealing title. It should be short. It should not have non-standard acronyms or abbreviations. It should not exceed two printed lines. It should include the name(s) and address (es) of all authors.

Abstract:

The summary should be two hundred words or less. It should briefly and clearly explain the key findings reported in the manuscript--must have precise statistics. It should not have abnormal acronyms or abbreviations. It should be logical in itself. Shun citing references at this point.

An abstract is a brief distinct paragraph summary of finished work or work in development. In a minute or less a reviewer can be taught the foundation behind the study, common approach to the problem, relevant results, and significant conclusions or new questions.

Write your summary when your paper is completed because how can you write the summary of anything which is not yet written? Wealth of terminology is very essential in abstract. Yet, use comprehensive sentences and do not let go readability for briefness. You can maintain it succinct by phrasing sentences so that they provide more than lone rationale. The author can at this moment go straight to shortening the outcome. Sum up the study, with the subsequent elements in any summary. Try to maintain the initial two items to no more than one ruling each.

- Reason of the study - theory, overall issue, purpose
- Fundamental goal
- To the point depiction of the research
- Consequences, including definite statistics - if the consequences are quantitative in nature, account quantitative data; results of any numerical analysis should be reported
- Significant conclusions or questions that track from the research(es)

Approach:

- Single section, and succinct
- As a outline of job done, it is always written in past tense
- A conceptual should situate on its own, and not submit to any other part of the paper such as a form or table
- Center on shortening results - bound background information to a verdict or two, if completely necessary
- What you account in an conceptual must be regular with what you reported in the manuscript
- Exact spelling, clearness of sentences and phrases, and appropriate reporting of quantities (proper units, important statistics) are just as significant in an abstract as they are anywhere else

Introduction:

The **Introduction** should "introduce" the manuscript. The reviewer should be presented with sufficient background information to be capable to comprehend and calculate the purpose of your study without having to submit to other works. The basis for the study should be offered. Give most important references but shun difficult to make a comprehensive appraisal of the topic. In the introduction, describe the problem visibly. If the problem is not acknowledged in a logical, reasonable way, the reviewer will have no attention in your result. Speak in common terms about techniques used to explain the problem, if needed, but do not present any particulars about the protocols here. Following approach can create a valuable beginning:

- Explain the value (significance) of the study
- Shield the model - why did you employ this particular system or method? What is its compensation? You strength remark on its appropriateness from a abstract point of vision as well as point out sensible reasons for using it.
- Present a justification. Status your particular theory (es) or aim(s), and describe the logic that led you to choose them.
- Very for a short time explain the tentative propose and how it skilled the declared objectives.

Approach:

- Use past tense except for when referring to recognized facts. After all, the manuscript will be submitted after the entire job is done.
- Sort out your thoughts; manufacture one key point with every section. If you make the four points listed above, you will need a least of four paragraphs.

- Present surroundings information only as desirable in order hold up a situation. The reviewer does not desire to read the whole thing you know about a topic.
- Shape the theory/purpose specifically - do not take a broad view.
- As always, give awareness to spelling, simplicity and correctness of sentences and phrases.

Procedures (Methods and Materials):

This part is supposed to be the easiest to carve if you have good skills. A sound written Procedures segment allows a capable scientist to replacement your results. Present precise information about your supplies. The suppliers and clarity of reagents can be helpful bits of information. Present methods in sequential order but linked methodologies can be grouped as a segment. Be concise when relating the protocols. Attempt for the least amount of information that would permit another capable scientist to spare your outcome but be cautious that vital information is integrated. The use of subheadings is suggested and ought to be synchronized with the results section. When a technique is used that has been well described in another object, mention the specific item describing a way but draw the basic principle while stating the situation. The purpose is to text all particular resources and broad procedures, so that another person may use some or all of the methods in one more study or referee the scientific value of your work. It is not to be a step by step report of the whole thing you did, nor is a methods section a set of orders.

Materials:

- Explain materials individually only if the study is so complex that it saves liberty this way.
- Embrace particular materials, and any tools or provisions that are not frequently found in laboratories.
- Do not take in frequently found.
- If use of a definite type of tools.
- Materials may be reported in a part section or else they may be recognized along with your measures.

Methods:

- Report the method (not particulars of each process that engaged the same methodology)
- Describe the method entirely
- To be succinct, present methods under headings dedicated to specific dealings or groups of measures
- Simplify - details how procedures were completed not how they were exclusively performed on a particular day.
- If well known procedures were used, account the procedure by name, possibly with reference, and that's all.

Approach:

- It is embarrassed or not possible to use vigorous voice when documenting methods with no using first person, which would focus the reviewer's interest on the researcher rather than the job. As a result when script up the methods most authors use third person passive voice.
- Use standard style in this and in every other part of the paper - avoid familiar lists, and use full sentences.

What to keep away from

- Resources and methods are not a set of information.
- Skip all descriptive information and surroundings - save it for the argument.
- Leave out information that is immaterial to a third party.

Results:

The principle of a results segment is to present and demonstrate your conclusion. Create this part a entirely objective details of the outcome, and save all understanding for the discussion.

The page length of this segment is set by the sum and types of data to be reported. Carry on to be to the point, by means of statistics and tables, if suitable, to present consequences most efficiently.You must obviously differentiate material that would usually be incorporated in a study editorial from any unprocessed data or additional appendix matter that would not be available. In fact, such matter should not be submitted at all except requested by the instructor.

Content

- Sum up your conclusion in text and demonstrate them, if suitable, with figures and tables.
- In manuscript, explain each of your consequences, point the reader to remarks that are most appropriate.
- Present a background, such as by describing the question that was addressed by creation an exacting study.
- Explain results of control experiments and comprise remarks that are not accessible in a prescribed figure or table, if appropriate.
- Examine your data, then prepare the analyzed (transformed) data in the form of a figure (graph), table, or in manuscript form.

What to stay away from

- Do not discuss or infer your outcome, report surroundings information, or try to explain anything.
- Not at all, take in raw data or intermediate calculations in a research manuscript.
- Do not present the similar data more than once.
- Manuscript should complement any figures or tables, not duplicate the identical information.
- Never confuse figures with tables - there is a difference.

Approach

- As forever, use past tense when you submit to your results, and put the whole thing in a reasonable order.
- Put figures and tables, appropriately numbered, in order at the end of the report
- If you desire, you may place your figures and tables properly within the text of your results part.

Figures and tables

- If you put figures and tables at the end of the details, make certain that they are visibly distinguished from any attach appendix materials, such as raw facts
- Despite of position, each figure must be numbered one after the other and complete with subtitle
- In spite of position, each table must be titled, numbered one after the other and complete with heading
- All figure and table must be adequately complete that it could situate on its own, divide from text

Discussion:

The Discussion is expected the trickiest segment to write and describe. A lot of papers submitted for journal are discarded based on problems with the Discussion. There is no head of state for how long a argument should be. Position your understanding of the outcome visibly to lead the reviewer through your conclusions, and then finish the paper with a summing up of the implication of the study. The purpose here is to offer an understanding of your results and hold up for all of your conclusions, using facts from your research and generally accepted information, if suitable. The implication of result should be visibly described. Infer your data in the conversation in suitable depth. This means that when you clarify an observable fact you must explain mechanisms that may account for the observation. If your results vary from your prospect, make clear why that may have happened. If your results agree, then explain the theory that the proof supported. It is never suitable to just state that the data approved with prospect, and let it drop at that.

- Make a decision if each premise is supported, discarded, or if you cannot make a conclusion with assurance. Do not just dismiss a study or part of a study as "uncertain."
- Research papers are not acknowledged if the work is imperfect. Draw what conclusions you can based upon the results that you have, and take care of the study as a finished work
- You may propose future guidelines, such as how the experiment might be personalized to accomplish a new idea.
- Give details all of your remarks as much as possible, focus on mechanisms.
- Make a decision if the tentative design sufficiently addressed the theory, and whether or not it was correctly restricted.
- Try to present substitute explanations if sensible alternatives be present.
- One research will not counter an overall question, so maintain the large picture in mind, where do you go next? The best studies unlock new avenues of study. What questions remain?
- Recommendations for detailed papers will offer supplementary suggestions.

Approach:

- When you refer to information, differentiate data generated by your own studies from available information
- Submit to work done by specific persons (including you) in past tense.
- Submit to generally acknowledged facts and main beliefs in present tense.

Administration Rules Listed Before
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Please carefully note down following rules and regulation before submitting your Research Paper to Global Journals Inc. (US):

Segment Draft and Final Research Paper: You have to strictly follow the template of research paper. If it is not done your paper may get rejected.

- The **major constraint** is that you must independently make all content, tables, graphs, and facts that are offered in the paper. You must write each part of the paper wholly on your own. The Peer-reviewers need to identify your own perceptive of the concepts in your own terms. NEVER extract straight from any foundation, and never rephrase someone else's analysis.

- Do not give permission to anyone else to "PROOFREAD" your manuscript.

- Methods to avoid Plagiarism is applied by us on every paper, if found guilty, you will be blacklisted by all of our collaborated research groups, your institution will be informed for this and strict legal actions will be taken immediately.)
- To guard yourself and others from possible illegal use please do not permit anyone right to use to your paper and files.

Criterion for Grading a Research Paper (Compilation) by Global Journals Inc. (US)

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Topics	Grades		
	A-B	**C-D**	**E-F**
Abstract	Clear and concise with appropriate content, Correct format. 200 words or below	Unclear summary and no specific data, Incorrect form Above 200 words	No specific data with ambiguous information Above 250 words
Introduction	Containing all background details with clear goal and appropriate details, flow specification, no grammar and spelling mistake, well organized sentence and paragraph, reference cited	Unclear and confusing data, appropriate format, grammar and spelling errors with unorganized matter	Out of place depth and content, hazy format
Methods and Procedures	Clear and to the point with well arranged paragraph, precision and accuracy of facts and figures, well organized subheads	Difficult to comprehend with embarrassed text, too much explanation but completed	Incorrect and unorganized structure with hazy meaning
Result	Well organized, Clear and specific, Correct units with precision, correct data, well structuring of paragraph, no grammar and spelling mistake	Complete and embarrassed text, difficult to comprehend	Irregular format with wrong facts and figures
Discussion	Well organized, meaningful specification, sound conclusion, logical and concise explanation, highly structured paragraph reference cited	Wordy, unclear conclusion, spurious	Conclusion is not cited, unorganized, difficult to comprehend
References	Complete and correct format, well organized	Beside the point, Incomplete	Wrong format and structuring

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