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Neural network prediction of the ultimate capacity of shear stud connectors on composite beams with profiled steel sheeting

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Abstract. In this paper, the efficiency of different Artificial Neural Networks (ANNs) in predicting the ultimate shear capacity of shear stud connectors is explored. Experimental data involving push-out test specimens of 118 composite beams from an existing database in the literature were used to develop the ANN model. The input parameters affecting the shear capacity were selected as sheeting, stud dimensions, slab dimensions, reinforcement in the slab and concrete compression strength. Each parameter was arranged in an input vector and a corresponding output vector, which includes the ultimate shear capacity of composite beams. For the experimental test results, the ANN models were trained and tested using three layered back-propagation methods. The prediction performance of the ANN was obtained. In addition to these, the paper presents a short review of the codes in relation to the design of composite beams. The accuracy of the codes in predicting the ultimate shear capacity of composite beams was also examined in a comparable way using the same test data. At the end of the study, the effect of all parameters is also discussed. The study concludes that all ANN models predict the ultimate shear capacity of beams better than codes.

1. Introduction

Composite structures consist of two or more parts of different materials attaching to each other to act as one. The advantage of composite action is that the desirable properties of each material can be used more efficiently. Shear connectors are used to achieve the connection between two materials, which are usually made of steel, and may have different shapes. Welded, headed shear studs are the most common type of shear connector used in the design of composite members nowadays, due to their rapid and easy construction [1].

In composite beams with profiled steel sheeting, many factors, such as the geometries and direction of profiled steel sheeting, the compressive strength of concrete, the reinforcement area and position, as well as the strength, dimension and location of shear connectors, affect the behavior of shear connectors. Push-out tests are commonly used to determine the capacity of the shear connectors and their load-slip behavior. According to Eurocode 4 [2], the push-out specimens consist of a steel beam section held in the vertical position by two identical concrete slabs. The concrete slabs are attached to the beam by shear connectors. The connection is subjected to a vertical

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load, which produces a shear load along the interface between the concrete slab and the beam flange on both sides. At a specified load or displacement, the slip between the slabs and the beam flange is evaluated. The failure load divided by the number of connectors is assumed as the shear connection capacity [3].

Composite construction, using steel and concrete, has been used since the early 1920s. It gained widespread use in bridges in the 1950s and in buildings in the 1960s [4]. Both push-out tests, which were first used in Switzerland in the 1930s [5], and full-scale beam tests have been used to develop shear stud strength prediction expressions. Push-out tests are usually used to evaluate a wide array of parameters because of the large size and expense of beam tests. The general setup of the test specimen and devices is given in Figure 1.

Early shear stud strength prediction equations were for solid slab construction, and the equations developed in the 1960s and 1970s were based on the results of push-out tests. The equations were modified for the use of steel deck in the late 1970s and were based on full-scale beam tests [6]. The stud strength equations given by Grant et al. [7] were developed from tests mostly using deck without stiffeners, where the studs were welded in the center of the deck rib. Beside the commonly used headed studs, to obtain optimum solutions for composite action, some investigations were undertaken with different types of welded shear connectors, like perfobond, T-connector, horseshoe, and bar connectors.

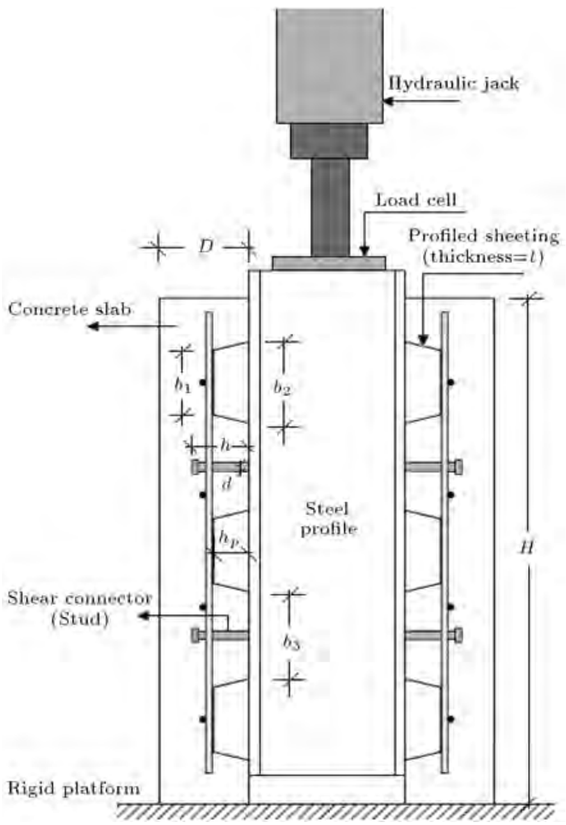

Figure 1. Test setup, dimension of concrete slab and steel sheeting [1].

There are many variables affecting the shear capacity of composite beams, such as sheeting type (width and depth of the rib of the profiled steel sheeting), stud dimensions (height and diameter), slab dimensions (width, depth and height), reinforcement in the slab, and concrete compression strength. The effect of these variables on the shear capacity of composite beams has been extensively studied and some empirical approaches have been developed related to the variables in the area of composite beams with perfobond ribs [5,8-10]. Galjaard and Walraven (2000) performed tests using shear studs, perfobond connectors, T-connectors and oscillating perfobond connectors, both with normal weight and lightweight concrete [11]. Johnson and Oehlers analyzed 125 push-out test results from 11 sources, performed 101 new push-out tests, and four composite T-beam tests, and performed a parametric study [12]. Also, Köroğlu conducted 4 push-out tests to study the behavior of Turkish extra seismic reinforcement steel bars as shear connectors in composite beams with profiled steel sheeting perpendicular to the beam. They also performed 4 push-out tests with the headed shear connectors as a shear connector to compare the Turkish extra seismic reinforcement steel bars versus headed shear connectors as a shear connector [13]. Vianna et al. studied neural network modeling of perfobond shear connector resistance for the first time. They also investigated perfobond shear connector capacity by a Bayesian neural network [14,15].

Because of the enormous variety of shear connectors, the strength and ductility of shear connectors are suggested to be determined experimentally. Thus, because of the fast automatic welding procedure, headed shear stud connectors are commonly used to ensure composite action. Since it is certainly the most investigated and understood form of shear connection, it is probably the most common form of welded shear connection.

The scope and objectives of the present work are:

a) To investigate the applicability of the Artificial Neural Network (ANN) in predicting the ultimate shear capacity of composite beams using experimental results collected from the literature;
b) To evaluate the accuracy of the building codes in predicting the shear strength of composite beams;
c) To compare the building code approaches and ANN results;
d) To discuss the effect of selected parameters on shear strength.

In this sense, the experimental data of 118 composite beams with headed shear stud connectors subjected to push-out tests were used from existing databases of Roddenbery [6], Köroğlu [13], Lloyd and Wright [16], and Kim et al. [17]. The experimental database is given in Table A1. Furthermore, some code approaches, such as AISC [18], Eurocode-4 [2], BSI-BS 5950 [19] and CSA [20], are also examined by comparing their predictions with the mentioned experimental study results. The results obtained by the proposed ANN model and the codes are compared to each other.

2. Calculating shear capacity of composite beams with profiled steel sheeting

The design strength and stiffness of composite beams with profiled steel sheeting depends on the shear connection behavior. According to experimental studies, the main factors defining the strength of shear connectors are given below. Also, a general view of the experimental set-up is also given in Figure 1.

a) Shape and dimensions of the shear connectors (h, d);
b) Quality of its material (f_u);
c) Concrete strength (f_{cu});
d) Type of load (static and dynamic);
e) Way of connecting the steel beams;
f) Distance between the shear connectors;
g) Dimensions of the concrete slab (B, H, D);
h) Percentage and way of reinforcing (area);
i) Sheeting type and dimension of steel sheeting (see Figure 1).

In the literature, several formulations have been proposed by various researchers. The review of some of these theories is given in Table 1.

Early tests by Fisher [21] were performed and several conclusions were drawn regarding the design of composite beams with formed metal decks. An equation for stud connector strength is given in Eq. (1), where b_0 is the average rib width, h_p is rib height, A_s is area of stud, f_c is the compressive strength of concrete and E_c is the Young modulus of concrete. When the ratio of rib width to height is greater than 1.75, the flexural strength of the beam can be developed with a full shear connection. Grant et al. [7] made a modification to the equation developed by Fisher [21], including the height effect of the shear stud connectors. They provided an empirical equation to calculate the shear capacity of headed shear studs in composite

Table 1. A Review of the regulations of shear capacity of composite beams.

Model	Expression	Number
Fisher [21]	$P_{\text{FISHER}} = 0.36\frac{b_0}{h_p} * 0.5A_s\sqrt{f_cE_c}$	(1)
Grant [7]	$P_{\text{GRANT}} = \frac{0.85}{\sqrt{N}}\left(\frac{b_0}{h_p}\right)\left[\left(\frac{h-h_p}{h_p}\right)0.5A_s\sqrt{f_cE_c}\right] \leq 0.5A_s\sqrt{f_cE_c}$	(2)
Hawkins and Mitchell [22]	$P_{\text{H\&M}} = \xi\lambda A_c\sqrt{f_c} \leq \left(4.1 - n^{-0.5}\right)A_c\left(\frac{E_c}{E_s}\right)^{0.4} f_{cu}^{0.35} f_u^{0.65}$	(3)
AISC [18]	$P_{\text{AISC}} = \underbrace{\left(\frac{0.85}{\sqrt{N}}\left(\frac{b_0}{h_p}\right)\left[\left(\frac{h}{h_p}\right) - 1.0\right]\right)}_{r_1} 0.5A_s\sqrt{f_cE_c} \leq A_sf_u$	(4)
BSI BS 5950 [19]	$P_{\text{BS5950}} = \left(0.25r_2\alpha d^2\sqrt{0.8f_cE_c}, 0.6r_2f_u\frac{\pi d^2}{4}\right)\text{min}$	(5)
EC 4 [2]	$P_{\text{EC4}} = \left(0.29r_3\alpha d^2\sqrt{f_cE_{cm}}, 0.8r_3f_u\frac{\pi d^2}{4}\right)\text{min}$	(6)
CSA [20]	$P_{\text{CSA}} = (4.2A_c\sqrt{f_c}, 0.5A_s\sqrt{f_cE_c} \leq A_sf_u)\,\text{min; for 76 mm deck}$ $P_{\text{CSA}} = (7.3A_c\sqrt{f_c}0.5A_s\sqrt{f_cE_c} \leq A_sf_u)\,\text{min; for 38 mm deck}$	(7a) (7b)

beams with profiled steel sheeting. Grant's expression for stud connector strength is given in Eq. (2), where N is the number of studs in a rib and h is the height of the stud. Hawkins and Mitchell [22] performed a linear regression analysis and developed two separate equations of shear connector shear strength due to concrete pull-out failure for a 76 mm deck and a 38 mm deck. In Eq. (3), for a 76 mm deck and a 38 mm deck, ξ is 0.35 and 0.61, respectively. The value of λ (factor dependent upon type of concrete) ranges from 0.75 and 1.0 and depends on the density of the concrete. In Eq. (3), n is the number of studs subjected to similar displacement, f_{cu} is the compressive cube strength of the concrete and f_u is the min. tensile strength of the stud. Rambo-Roddenberry (2002) [6] carried out 92 push-out tests to study the behavior of headed shear stud connectors in composite beams with profiled steel sheeting perpendicular to the beam. He also provided a new strength prediction model based on the strength prediction equations to calculate the shear capacity of headed shear studs. In his approaches, the strength prediction divided four parts that differ from each other to the d/t ratio and stud height.

The design strength and stiffness of composite beams with profiled steel sheeting depend on the shear connection behavior. Because of the steel deck geometry of the composite beams with profiled steel sheeting, the strength of the shear connectors may be reduced. An empirical expression for this reduction was developed by evaluating the results of the composite beam tests in many standards.

The AISC equation [18] for the calculation of the design strength of headed shear stud connectors in composite beams with profiled steel sheeting perpendicular to the steel beam is given Eq. (4). The r_1 (reduction factor), which should not be taken greater than 1.0, is a function of the deck geometry and the number of studs in a rib. The elastic modulus of concrete is $E_c = 4700\sqrt{f_c}$ according to the ACI building code [23]. In the BSI (BS 5950 Part 3) [19], the design strength of the headed shear stud connector in composite beams with profiled steel sheeting perpendicular to the steel beam is determined by multiplying the values by the reduction factor, as given in Eq. (5). In the expression where the r_2 reduction factor ($r_2 \leq 1.0$) is calculated from this equation:

$$\underbrace{\left(\frac{0.85}{\sqrt{N}}\left(\frac{b_0}{h_p}\right)\left[\left(\frac{h}{h_p}\right)-1.0\right]\right)}_{r_2},$$

by using ($N = 1$). The design strength for EC4 [2] of the headed stud in composite beams with profiled steel sheeting perpendicular to the steel beam is similar to the AISC equations [18], except the constant 0.5 is changed to 0.29 in the equation, and the upper limit on this strength is 80% of the tensile strength of the stud. In the EC4 expression, if $3 \leq \frac{h}{d} \leq 4$, α is $\left(\frac{h}{d}-1\right)$ and $\alpha = 1$ for $\frac{h}{d} > 4$. The strength reduction factor (r_3) ranges from 1.0 to 0.6, and is calculated using r_2, but replacing the factor 0.85 with 0.7. The CSA specification [20] is the same equation as the one in the AISC specification [18]. According to the CSA [20], the strength of the headed shear stud connector depends on the depth of the rib, as given in Eqs. (7a) and (7b).

3. Selection of database (description of data)

The experimental data of composite beams with headed shear stud connectors subjected to push-out tests were used from the existing databases of Roddenbery [6], Lloyd and Wright [16], Kim et al. [17], and Köroğlu [13]. A push-out test specimen consists of a short steel beam section held in a vertical position by two similarly reinforced concrete slabs attached to the beam flanges by shear connectors, as shown in Figure 1. The overall system is subjected to a vertical load to produce a shear load along the interface between the concrete slab and the beam flange on both sides using a hydraulic jack. Instead of full scale composite beam tests, push-out tests are conducted to study the effect of using shear studs in composite beams because of the easy and fast manufacturing of test specimens. It has been shown from several hundred tests that push-out tests can be used to quantitatively assess the strength of shear studs for composite beams.

In this study, the test specimens were solid rectangular slabs with profiled steel sheeting, subjected to a pure axial load. The compressive strength of concrete ranged from 20.1 to 48.81 MPa, the stud diameter between 12.7 and 19 mm, the stud height ranged between 65 and 127 mm, with a slab width of 450 to 1350 mm, the slab depth ranged between 75 mm to 150 mm, with a slab height between 425 and 914 mm; reinforcement in the slab is between 0 and 193 mm^2, and the steel sheeting, b_1, b_2 and b_3, ranged from 114 to 140 mm, 159 to 191 mm and 35 to 127 mm, respectively.

The complete list of the data is given in Table A1. As seen from Table A1, a total of 46 tests are used to satisfy the variables mentioned above. Specimens are identified using the notations in the first row, with the first letter of the researchers' names. Some data from the tests were not used because of obtaining the same criteria.

3.1. Lessons learned from existing experimental studies

- Using a shallow slab when the concrete cover is smaller above the stud may cause cracking on the concrete surface at lower loads, due to the concentration of shear force near the head of the stud.

- The low stiffness of the concrete led to a reduction in the connection resistance of the stud. Thus, it is recommended to use mesh reinforcement in the slabs with low strength concrete. The split failure of concrete can occur when the concrete has low compressive strength. Because the strength of the stud in push-out tests depends on the material properties of both steel and concrete, for the slab with high strength concrete, failure can occur in the stud.
- When failure occurs because of the properties of the stud, the diameter and tensile strength of the shear stud connector is essential. According to EC4 [2] and BSI [19], the tensile strength of the shear stud connector is not taken greater than 450 N/mm^2.

4. Fundamental aspects of Neural Networks (NNs)

A neural network is a 'machine' that is designed to model the way in which the brain performs a particular task or function of interest, and the network is usually implemented using electronic components or simulated in software on a digital computer. Neural networks are an information processing technique built on processing elements, called neurons, which are connected to each other [24].

An artificial neuron is the basic element of a neural network, which consists of three main components, namely, weights, bias, and an activation function, where:

$$u_i = \sum_{j=1}^{H} w_{ij} x_j + b_i. \tag{8}$$

Summation u_i is transformed as the output using a scalar-to-scalar function called an "activation or transfer function" as follows:

$$O = f(u_i). \tag{9}$$

Neural networks are commonly classified by their network topology (i.e. feedback, feed forward) and learning or training algorithms (i.e. supervised, unsupervised). For example, a multilayer feed forward neural network with back propagation indicates the architecture and learning algorithm of the neural network. The back propagation algorithm is used in this study, which is the most widely used supervised training method for training multilayer neural networks, due to its simplicity and applicability. It is based on the generalized delta rule and was popularized by Rumelhart et al. [25].

4.1. Optimal NN model selection

The performance of a NN model mainly depends on the network architecture and parameter settings. One of the most difficult tasks in NN studies is to find this optimal network architecture, which is based on determining the number of optimal layers and neurons in the hidden layers by a trial and error approach. The assignment of initial weights and other related parameters may also influence the performance of the NN to a great extent. However, there is no well-defined rule or procedure to obtain optimal network architecture and parameter settings where the trial and error method still remains valid. This process is very time consuming.

In this study, the Matlab NN toolbox is used for NN applications. Various back propagation training algorithms used are given in Table 2. The Matlab NN toolbox randomly assigns the initial weights for each run, each time, which considerably changes the performance of the trained NN, even when all parameters and NN architecture are kept constant. This leads to extra difficulties in the selection of optimal network architecture and parameter settings. To overcome this difficulty, a program has been developed in Matlab that handles the trial and error process automatically. The program tries various numbers of layers and neurons in the hidden layers, both for first and second hidden

Table 2. Back propagation training algorithms used in NN training.

MATLAB function name	Algorithm
trainbfg	BFGS quasi-Newton back propagation
traincgf	Fletcher-Powell conjugate gradient back propagation
traincgp	Polak-Ribiere conjugate gradient back propagation
traingd	Gradient descent back propagation
traingda	Gradient descent with adaptive lr back propagation
traingdx	Gradient descent w/momentum & adaptive linear back propagation
trainlm	Levenberg-Marquardt back propagation
trainoss	One step secant back propagation
trainrp	Resilient back propagation (Rprop)
trainscg	Scaled conjugate gradient back propagation

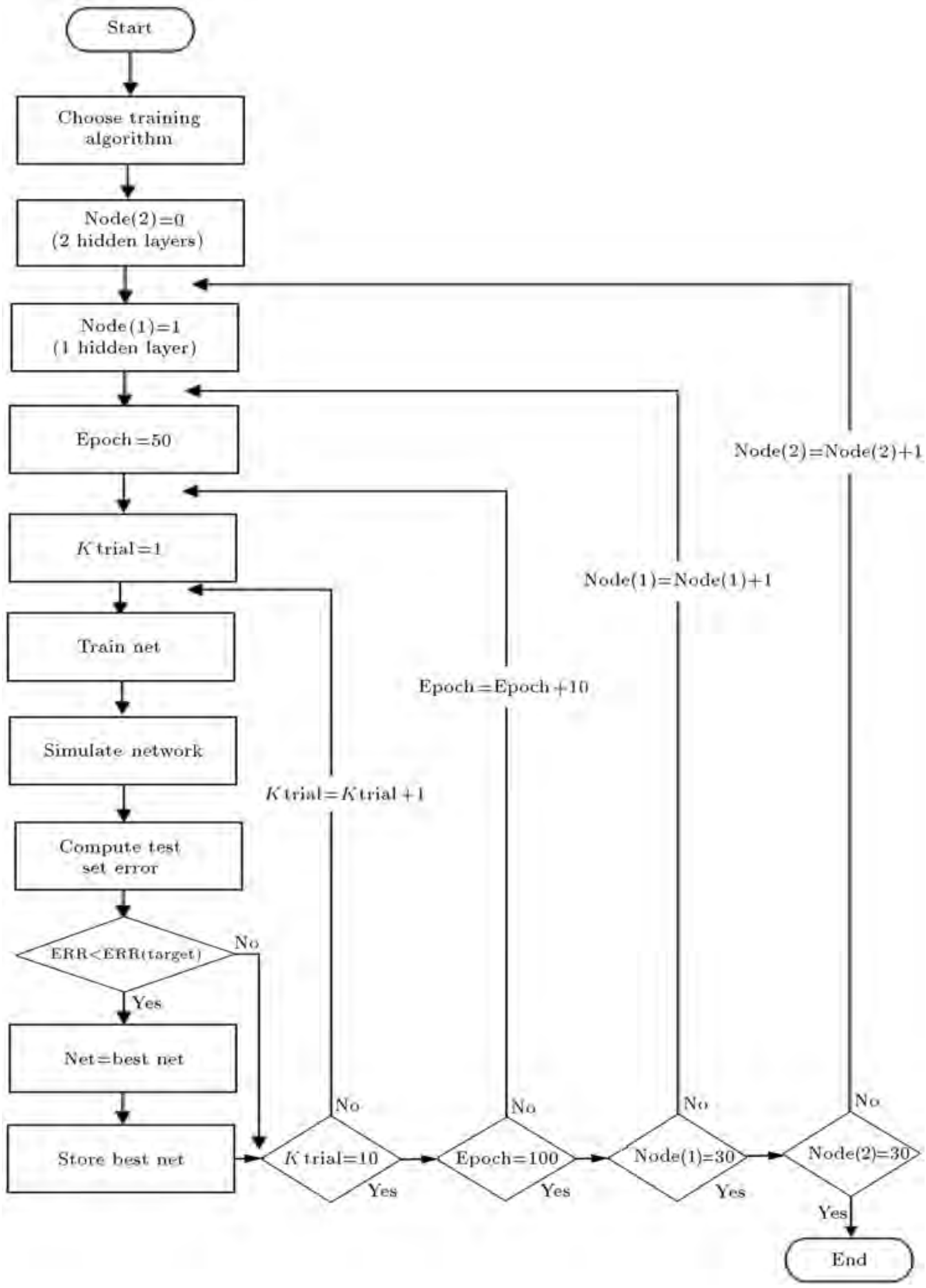

Figure 2. Flowchart of whole process.

layers, for a constant epoch, for several times, and selects the best NN architecture with the minimum MAPE (Mean Absolute % Error) or RMSE (Root Mean Squared Error) of the testing set, as the training of the testing set is more critical. For instance, a NN architecture with 1 hidden layer with 7 nodes is tested 10 times and the best NN is stored, where, in the second cycle, the number of hidden nodes is increased up to 8 and the process is repeated. The best NN for cycle 8 is compared with cycle 7, and the best one is stored as the best NN. This process is repeated N times, where N denotes the number of hidden nodes for the first hidden layer. This whole process is repeated for the changing number of nodes in the second hidden layer. Moreover, this selection process is performed for different back propagation training algorithms available in the literature, shown in Table 2. The program begins with the simplest NN architecture, i.e. the NN with 1 hidden node for the first and second hidden layers and resulting in optimal NN architecture. The flowchart of the whole process is shown in Figure 2. This algorithm has been

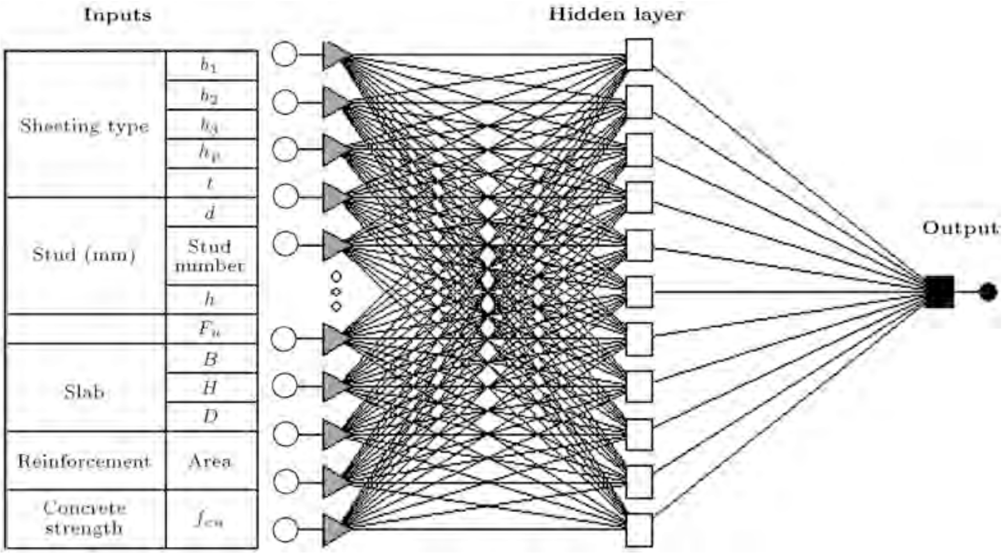

Figure 3. Optimum NN architecture (14-11-1).

successfully applied to a wide range of neural network applications [26-29].

4.2. ANN-based models for calculating shear capacity of composite beams with profiled steel sheeting

The main focus of this study is the explicit formulation of the ultimate shear capacity of composite beams with profiled steel sheeting using neural networks based on the experimental database given in Table A1. The ultimate shear capacity will be obtained as a function of sheeting type (width and depth of rib of the profiled steel sheeting), stud dimensions (height and diameter), slab dimensions (width, depth, and height), reinforcement in the slab and concrete compression strength.

As shown in Figure 3, 14 different input parameters were used to model the ultimate shear capacity of composite beams with profiled steel sheeting. In the development of the NN model, a set consisting of 46 tests were used that were obtained from the literature shown in the experimental database in Table A1. The database was divided into training (80%) and testing (20%) sets. The performance of the algorithm has been checked by using a testing algorithm. The optimal NN architecture was found to be 14-11-1, i.e. the NN model with 11 hidden neurons shown in Figure 3. The training algorithm was Levenberg-Marquardt back propagation. Hyperbolic tangent sigmoid and Log sigmoid transfer functions were used for the hidden and output layers, respectively. Statistical parameters of testing and training sets, and the overall results of NN models, are presented in Table 3. The NN results versus actual test results are presented in Figure 4. NN results are observed to be very close to actual test results.

Table 3. Statistical parameters of testing and training sets and overall results of NN model.

	Mean	COV	R^2
Testing set	0.99	0.15	0.93
Training set	1.00	0.08	0.96
Overall	1.01	0.10	0.95

5. Explicit formulation of the NN model and comparison to code equations

NN applications are treated as black-box applications, in general. However, this study opens this black box and introduces the NN application in a closed form solution. Using the weights and biases of a trained NN model, rotation can be computed as follows:

$$P(kN) = 111.35\left(\frac{1}{1+e^{-W}}\right), \tag{10}$$

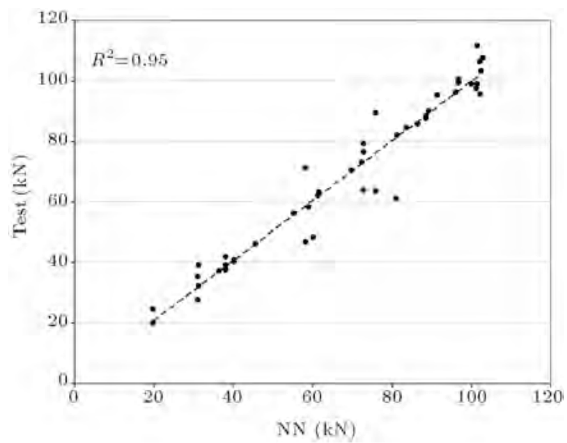

Figure 4. Comparison between NN and experimental results.

where $\tanh(x) = (e^x - e^{-x})/(e^x + e^{-x})$, and, finally, output is computed as:

$$W = \left[-1.50 * \left(\frac{1}{1+e^{-U_1}}\right) + 0.19 * \left(\frac{1}{1+e^{-U_2}}\right) + 0.92 * \left(\frac{1}{1+e^{-U_3}}\right) + 0.92 * \left(\frac{1}{1+e^{-U_4}}\right) + 4.09 * \left(\frac{1}{1+e^{-U_5}}\right) - 3.58 * \left(\frac{1}{1+e^{-U_6}}\right) + 4.37 * \left(\frac{1}{1+e^{-U_7}}\right) + 2.18 * \left(\frac{1}{1+e^{-U_8}}\right) - 4.36 * \left(\frac{1}{1+e^{-U_9}}\right) - 2.17 * \left(\frac{1}{1+e^{-U_{10}}}\right) + 2.80 * \left(\frac{1}{1+e^{-U_{11}}}\right) \right] - 0.61,$$

$$U_1 = (-0.0316 * b_1) + (-0.0011 * b_2) + (0.0088 * b_3) + (0.0255 * h_p) + (-2.2855 * t) + (-0.0497 * d) + (-1.2927 * SN) + (0.0092 * h) + (-0.0068 * F_u) + (-0.0018 * B) + (0.0028 * H) + (-0.0052 * D) + (-0.0006 * A) + (-0.0386 * f_{cu}) + 11.58,$$

$$U_2 = (-0.0549 * b_1) + (-0.0105 * b_2) + (-0.0144 * b_3) + (-0.0069 * h_p) + (0.2289 * t) + (-0.0499 * d) + (-0.5421 * SN) + (0.0078 * h) + (-0.0143 * F_u) + (-0.0001 * B) + (-0.0011 * H) + (-0.0029 * D) + (-0.009 * A) + (0.0457 * f_{cu}) + 20.51,$$

$$U_3 = (-0.0492 * b_1) + (0.0051 * b_2) + (0.0095 * b_3) + (-0.0188 * h_p) + (1.957 * t) + (-0.0708 * d) + (1.1737 * SN) + (-0.0247 * h) + (0.0022 * F_u) + (0.0003 * B) + (0.0007 * H) + (-0.019 * D) + (-0.0013 * A) + (0.0261 * f_{cu}) + 8.69,$$

$$U_4 = (-0.0514 * b_1) + (-0.0269 * b_2) + (0.0087 * b_3) + (0.0111 * h_p) + (-2.3629 * t) + (-0.0964 * d) + (0.3852 * SN) + (-0.0002 * h) + (-0.0084 * F_u) + (0.0007 * B) + (0.0035 * H) + (-0.0037 * D) + (0.0022 * A) + (0.0103 * f_{cu}) + 14.27,$$

$$U_5 = (-0.0364 * b_1) + (-0.0364 * b_2) + (0.0075 * b_3) + (0.0307 * h_p) + (-1.4289 * t) + (0.2057 * d) + (-01119 * SN) + (-0.0068 * h) + (0.0142 * F_u) + (0.001 * B) + (-0.002 * H) + (-0.0158 * D) + (0.0051 * A) + (-0.0509 * f_{cu}) + 4.58,$$

$$U_6 = (-0.0025 * b_1) + (-0.0013 * b_2) + (0.0003 * b_3) + (0.0345 * h_p) + (-2.2717 * t) + (0.1276 * d) + (-0.0526 * SN) + (-0.0195 * h) + (0.0141 * F_u) + (0.0005 * B) + (-0.0013 * H) + (0.0099 * D) + (-0.0064 * A) + (-0.01852 * f_{cu}) + 2.52,$$

$$U_7 = (-0.043 * b_1) + (-0.0259 * b_2) + (-0.0065 * b_3) + (0.0112 * h_p) + (-0.0006 * t) + (0.0703 * d)$$

$$+(-0.595*SN)+(-0.0164*h)$$

$$+(-0.0052*F_u)+(0.0008*B)$$

$$+(-0.0012*H)+(0.0175*D)$$

$$+(0.0121*A)+(-0.0547*f_{cu})+17.25,$$

$$U_8=(0.0083*b_1)+(-0.0495*b_2)$$

$$+(0.0062*b_3)+(-0.0197*h_p)$$

$$+(-0.9012*t)+(0.0361*d)$$

$$+(-0.1816*SN)+(-0.0182*h)$$

$$+(-0.0163*F_u)+(0.0005*B)$$

$$+(0.0022*H)+(-0.0002*D)$$

$$+(0.0086*A)+(0.059*f_{cu})+11.2,$$

$$U_9=(0.0455*b_1)+(0.0469*b_2)$$

$$+(-0.0107*b_3)+(0.0284*h_p)$$

$$+(-0.1459*t)+(-0.1458*d)$$

$$+(0.1339*SN)+(0.0029*h)$$

$$+(0.0069*F_u)+(-0.0001*B)$$

$$+(-0.0012*H)+(0.0096*D)$$

$$+(-0.0067*A)+(0.1268*f_{cu})-21.12,$$

$$U_{10}=(0.0547*b_1)+(-0.0075*b_2)$$

$$+(-0.012*b_3)+(-0.0081*h_p)$$

$$+(-3.1565*t)+(-0.0496*d)$$

$$+(0.9817*SN)+(-0.0062*h)$$

$$+(0.0116*F_u)+(-0.0018*B)$$

$$+(-0.0022*H)+(-0.0127*D)$$

$$+(-0.001*A)+(0.0186*f_{cu})-3.89,$$

$$U_{11}=(-0.0293*b_1)+(0.0409*b_2)$$

$$+(0.0135*b_3)+(0.0366*h_p)$$

$$+(0.2630*t)+(-0.0826*d)$$

$$+(0.0297*SN)+(0.0116*h)$$

$$+(0.0141*F_u)+(-0.0012*B)$$

$$+(-0.0017*H)+(-0.0037*D)$$

$$+(0.0049*A)+(0.0127*f_{cu})-12.8,$$

The predicted ultimate shear strength values of the NN model are observed to be in agreement with the experimental data. In order to investigate the accuracy of the standards for the shear capacity of composite beams with profiled steel sheeting, the test results given in Table A1 were compared with the approaches of the mentioned codes. In Table 4, performance evaluation of the ANN and code approaches is given, with respect to predicting the capability of the shear strength of composite beams. As seen, the proposed NN model is, by far, more accurate than available design codes. Also, in Table A1, the results of code approaches and ANN are given with separate columns for the 118 specimens. The AISC [18], BSI [19], EC4 [2] and CSA [20] have very close estimation capacities of shear strength. The R^2 coefficients are between 76.45% and 78.33%.

Training error, test error, training time and correlation coefficient (R^2) have been used for the initial performance evaluation of different back propagation training algorithms in the literature. In this study, correlation coefficient (R^2) has been chosen as the performance criteria, and the R^2 value is obtained about 95%. Therefore, the study has shown the feasibility of the potential use of the ANN model on the ultimate capacity of composite beams.

In Figure 5, the analyses of shear capacity performance change depending on the profiled steel sheeting given. Changing the parameters of the profiled steel sheeting (b_1, b_2, and b_3) shows whether the slab is solid or not. The figures show that making the slab more similar to solid increases the ultimate shear capacity. The effect of the other parameters, such as t or h_p, on the ultimate shear capacity of composite beams with profiled steel sheeting can be seen in Figure 5.

6. Summary and conclusion

The purpose of the research reported herein is to predict the ultimate shear capacity of composite beams

Table 4. Comparison of accuracy of NN model versus various design codes.

	NN	P_{AISC}	P_{EC4}	P_{BS}	P_{CSA}
Mean	1.01	0.72	0.91	1.20	0.72
COV	0.10	0.17	0.19	0.18	0.17
R2	0.95	0.7645	0.7732	0.7795	0.7833

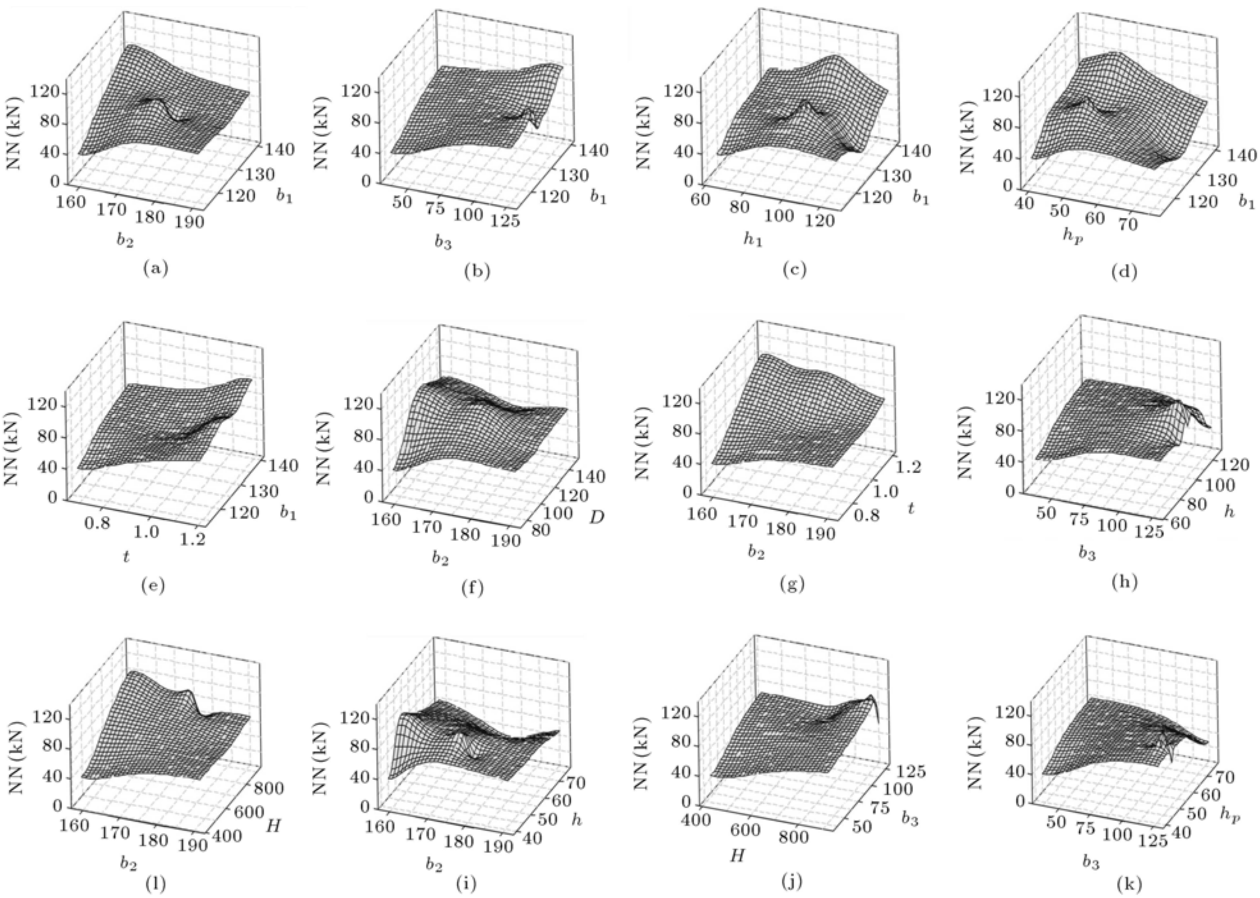

Figure 5. The effect of parameters on the ultimate shear capacity of composite beams with profiled steel sheeting.

with profiled steel sheeting by Artificial Neural Networks (ANNs). In the study, the effect of the selected design parameters of composite beams was also discussed. The 46 composite beam test database used for NN training is based on experimental results from literature. A part of Köroğlu's thesis and calculating the shear capacity of composite beams with profiled steel sheeting was discussed. The behavior of the shear connection between steel beams and composite slabs using through-deck welded shear connectors was studied here. Back propagation NN are used for the training process. The proposed ANN model shows perfect agreement with experimental results ($R^2 = 0.95$). The selected algorithm has shown stronger estimation power than the building code approaches. In this study, R^2 values, as obtained, were approximately 95%. Therefore, the study has shown the feasibility of the potential use of the ANN model on the ultimate capacity of composite beams.

The code approaches (such as AISC, BSI, EC4 and CSA) on the ultimate capacity of stud shear connectors on composite beams with profiled steel sheeting have quite low close estimation capacities of shear strength. By using the code formulation, R^2 coefficients have been calculated between 76.45% and 78.33%. On the other hand, the result of the algorithm has been found to be at a satisfactory level when compared with the current codes, which are very limited in the prediction of the ultimate capacity of stud shear connectors on composite beams with profiled steel sheeting.

It is obvious from statistical results above that the proposed ANN model accurately learned to map the relationship between the ultimate shear capacities of composite beams with profiled sheeting. Also, it is clear that the results obtained by the codes and ANN approaches are limited with the selected dataset given in the text.

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Nomenclature

A	Area of stud shank
A_c	Concrete pull-out failure surface area
A_s	The cross-sectional area of the headed stud shear connector
B	Width of composite concrete slab
b_0	Average width of concrete rib of the profiled steel sheeting
b_1	Smaller width of rib of the profiled steel sheeting
b_2	Larger width of rib of the profiled steel sheeting
b_3	Upper section of smaller width of rib of the profiled steel sheeting
D	Depth of composite concrete slab
d	Diameter of headed stud shear connector
E_c	Initial Young's modulus of concrete
E_{cm}	Mean value of the secant modulus tabulated in the EC4
e	Distance from the center of the stud's longitude
f_c	Compressive cylinder strength of concrete
f_{cu}	Compressive cube strength of concrete
f_u	Minimum specified tensile stress of the stud shear connector
f_{ys}	Yield stress of headed stud shear connector
H	Height of composite concrete slab
h	Height of the headed stud
h_p	Depth of the rib
N	Number of studs in one rib of the profiled steel sheeting
n	Number of studs subjected to similar displacements
P_{AISC}	Design strength calculated using the American Specification
$P_{\mathrm{BS\ 5950}}$	Design strength calculated using British Standard
P_{CSA}	Design strength calculated using Canadian Standards Association
P_{EC4}	Design strength calculated using European Code
P_{FISHER}	Design strength calculated using Fisher formula
P_{GRANT}	Design strength calculated using grant formula in solid slab
P_{OOLGAARD}	Design strength calculated using Oolgaard formula
P_{POS}	Concrete pull-out strength of a stud in a composite slab
P_{RR}	Design strength calculated using Rambo-Roddenbery formula
P_{SOL}	Design strength calculated using Fisher formula in solid slab
r	Reduction factor
r_1	Reduction factor
r_2	Reduction factor
r_3	Reduction factor
V_c	Shear strength due to concrete pull-out failure (N)
λ	Factor dependent upon type of concrete
t	Profiled steel sheeting thickness

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Appendix

Experimental data of push out tests were used from existing databases of Roddenbery [6], Lioyd and Wright [16], Kim et al. [17] and Köken and Köroğlu [13] as shown in Table A1.

Biographies

Mehmet Alpaslan Köroğlu obtained a BS degree in Civil Engineering from Gaziantep University, Turkey, in 2004, and MS and PhD degrees in Structural Engineering from Selcuk University, Turkey, in 2007 and 2012, respectively. He is currently Assistant Professor in the Civil Engineering Department of Necmettin Erbakan University. His research interests include: Design of strengthened reinforced concrete structures, steel beam-column connections and composite beams.

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Table A1. Experimental database [6,13,16,17].

	Specimen	Stud Position	Sheeting Type					Stud				Slab			Reinforc-ement	Concrete Strenght	P per stud (KN)					
		Strong, Weak or Medium	b_1 (mm)	b_2 (mm)	b_3 (mm)	h_p (mm)	t (mm)	d (mm)	Stud no.	h (mm)	F_u (Mpa)	B (mm)	H (mm)	D (mm)	mm^2	F_{cu} (Mpa)	P_E	P_{NN}	P_{NN}	P_{EC4}	P_{BS}	P_{CSA}
Roddenbery [6]	S1	S	127	178	127	50	0.914	12.7	1	100	510	914	914	150	0	30.54	38.97	31.55	56.31	45.58	34.19	56.38
	S2	2S	127	178	127	50	0.914	12.7	2	100	510	914	914	150	0	30.54	36.87	36.74	56.31	37.26	34.19	56.38
	S3	W	127	178	127	50	0.914	12.7	1	100	510	914	914	150	0	30.54	32.04	31.55	56.31	45.58	34.19	56.38
	S4	S	127	178	127	50	0.914	15.88	1	100	500	914	914	150	0	20.1	61.74	61.41	63.61	53.21	39.35	64.41
	S5	W	127	178	127	50	0.914	15.88	1	89	500	914	914	150	0	20.1	55.80	55.43	63.61	53.21	39.35	64.41
	S6	2S	127	178	127	50	0.914	15.88	2	100	500	914	914	150	0	20.1	60.70	81.28	63.61	42.57	39.35	64.41
	S7	S	127	178	127	50	0.914	12.7	1	100	510	914	914	150	0	40.61	40.03	40.28	64.67	45.58	34.19	64.57
	S8	2S	127	178	127	50	0.914	12.7	2	100	510	914	914	150	0	40.61	45.95	45.83	64.67	45.58	34.19	64.57
	S9	W	127	178	127	50	0.914	12.7	1	100	510	914	914	150	0	40.61	40.79	40.28	64.67	45.58	34.19	64.57
	S10	S	127	178	127	50	0.914	19	1	100	447	914	914	150	0	48.81	89.01	76.11	126.67	101.34	76.00	126.67
	S11	2S	127	178	127	50	0.914	19	2	100	447	914	914	150	0	48.81	85.36	86.70	126.67	101.34	76.00	126.67
	S12	W	127	178	127	50	0.914	19	1	100	447	914	914	150	0	48.81	63.39	76.11	126.67	101.34	76.00	126.67
	S13	S	127	178	127	50	0.914	15.88	1	100	500	914	914	150	0	32.47	70.85	58.53	92.65	71.26	53.45	92.30
	S14	2S	127	178	127	50	0.914	15.88	2	100	500	914	914	150	0	32.47	72.77	72.56	92.65	71.26	53.45	92.30
	S15	W	127	178	127	50	0.914	15.88	1	100	500	914	914	150	0	32.47	46.45	58.53	92.65	71.26	53.45	92.30
	S16	S	127	178	127	50	0.914	9.53	1	100	532	914	914	150	0	27.1	24.36	19.85	29.49	23.98	17.73	29.03
	S17	S	127	178	127	50	0.914	22.23	1	100	464	914	914	150	0	27.1	84.16	83.73	157.94	130.47	96.49	157.94
	S18	W	127	178	127	50	0.914	9.53	1	100	532	914	914	150	0	27.1	19.86	19.85	29.49	23.98	17.73	29.03
	S20	S	127	178	127	76	0.914	9.53	1	127	532	914	914	150	0	36.13	35.04	31.30	35.72	25.67	19.25	36.01
	S22	W	127	178	127	76	0.914	9.53	1	127	532	914	914	150	0	36.13	27.50	31.30	35.72	25.67	19.25	36.01
	S26	S	127	178	127	50	0.914	19	1	100	447	914	914	150	0	34.47	100.26	97.08	126.67	101.34	76.00	108.03
	S27	S	127	178	127	50	0.914	19	1	100	447	914	914	150	0	34.47	99.14	97.08	126.67	101.34	76.00	108.03
	S28	W	127	178	127	50	0.914	19	1	100	447	914	914	150	0	32.34	63.54	73.03	126.67	101.34	76.00	108.03
	S29	W	127	178	127	50	0.914	19	1	100	447	914	914	150	0	32.34	76.23	73.03	126.67	101.34	76.00	108.03
	S30	S	127	178	127	50	0.914	19	1	100	447	914	914	150	0	32.34	78.85	73.03	126.67	101.34	76.00	126.67
Lloyd & Wright [16]	1S1	S	125	175	125	50	1.2	19	1	100	448	450	900	115	98	44.8	95.26	102.45	126.96	101.57	76.17	126.96
	2S2	S	125	175	125	50	1.2	19	1	100	448	675	900	115	98	35.3	81.80	81.49	126.96	101.57	76.17	126.96
	3S3	S	125	175	125	50	1.2	19	1	100	448	900	900	115	98	39.5	89.89	89.44	126.96	101.57	76.17	126.96
	4S4	S	125	175	125	50	1.2	19	1	100	448	1125	900	115	98	46.3	95.84	96.27	126.96	101.57	76.17	126.96
	5S5	S	125	175	125	50	1.2	19	1	100	448	1350	900	115	98	43.6	102.90	102.54	126.96	101.57	76.17	126.96
	6S6	S	125	175	125	50	1.2	19	1	100	448	900	900	115	193	43.8	98.76	100.35	126.96	101.57	76.17	126.96
	7S7	S	125	175	125	50	1.2	19	1	100	448	900	900	115	142	37.3	94.89	91.49	126.96	101.57	76.17	126.96
	8S8	S	125	175	125	50	1.2	19	1	100	448	900	900	115	142	39.6	87.27	88.88	126.96	101.57	76.17	126.96
	9S9	S	125	175	125	50	1.2	19	1	100	448	900	900	115	142	39.8	88.37	88.68	126.96	101.57	76.17	126.96
	10S10	S	140	160	120	50	1.2	19	1	100	448	900	900	115	142	38.3	97.23	101.53	126.96	101.57	76.17	126.96
	11A1	S	140	160	120	50	1.2	19	1	100	448	900	900	115	142	38.5	111.35	101.61	126.96	101.57	76.17	126.96
	12A2	S	140	160	120	50	1.2	19	1	100	448	900	900	115	142	40.2	106.01	102.46	126.96	101.57	76.17	126.96
	13A3	S	140	160	120	50	1.2	19	1	100	448	900	900	115	142	38.5	98.84	101.61	126.96	101.57	76.17	126.96
	14A4	S	140	160	120	50	1.2	19	1	100	448	900	900	115	142	41.2	107.23	103.07	126.96	101.57	76.17	126.96
Kim et al [17]	1	M	114	159	35	40	0.68	13	1	65	435	450	425	75	142	34.5	41.50	38.36	50.10	44.10	34.63	57.71
	2	M	114	159	35	40	0.68	13	1	65	435	450	425	75	142	34.5	38.75	38.36	50.10	44.10	34.63	57.71
	3	M	114	159	35	40	0.68	13	1	65	435	450	425	75	142	34.5	37.25	38.36	50.10	44.10	34.63	57.71
Köroğlu [13]	2K10	W	129	191	116	52	1	19	1	80	510	650	700	100	0	25.3	58.00	59.12	109.58	90.52	66.95	109.58
	1K10	W	129	191	116	52	1	19	1	80	510	650	700	100	0	26.1	63.00	61.78	112.17	92.66	68.53	112.17
	1K12	S	129	191	116	52	1	19	1	80	510	650	700	120	0	27.4	70.00	70.07	116.33	96.10	71.07	116.33
	2K12	S	129	191	116	52	1	19	1	80	510	650	700	120	0	20.1	48.00	60.43	92.21	76.17	56.34	92.21