

Contribution of Steel Shear Reinforcement in Two-Way Shear Resistance of Flat Plates (Parametric Study)

Mohamed M. Badawy¹

¹Civil Engineering Department, High Institute for Engineering and Technology Al-Obour,. Egypt

Corresponding Author Email: mohamedm@oi.edu.eg, Mohamed_badawy80@yahoo.com

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Abstract – The current research presents an experimental, analytical and finite element investigation to study shear enhancement of flat plates by shear heads. Experimental results of nine half-scale concrete flat plate specimens were tested. Three different column aspect ratio were tested in the current study as follows: 1:1, 2:1 and 4:1. Two different lengths of shear heads from column face equal to $2.00d$ and $2.60d$ were examined with change of column aspect ratios in which "d" is flat plate depth. Structural behaviors were evaluated in terms of overall load-deflection response, ultimate loading capacity and ductility. Failure patterns and strain distribution were also discussed. The research also presents parametric study through numerical analysis taken in to account most important variables which haven't been studied in the practical program. Different arrangement of shear head legs, changing cubic compressive strength of concrete were some of these variables. Several different lengths of shear heads were also studied. Experimental and numerical results were analyzed through regression analysis to propose a new formula to predict the ultimate punching shear capacity of flat plates with shear heads.

Keywords – Flat plates; Shear heads; Column aspect ratio; Finite element investigation; Parametric study; Numerical analysis; Regression analysis

INTRODUCTION

Punching shear is one of the most important criteria that control design of flat plates. There are many traditional ways to enhance punching shear resistance of flat plates such that using thick slab and large column or drop panel utilization with and without column heads. The ACI 318 and CSA (Canadian Standards) codes are based mainly on Moe's work (1961), while the BS 8110 (British Standard Institution) and EC 2 codes are based on Regan's work (1974). The present building code specifications for the shear strength of reinforced concrete slabs are based on the test results of plates made with relatively low compressive strengths, varying mostly from 14 to 40 MPa. The Kinnunen and Nylander concrete punching rational model (1960) still provides one of the best accounts of the punching behavior of concrete plates without shear reinforcement. Modern European codes of practice treat punching shear in terms of shear stresses calculated at control perimeters located at relatively large distance from the column or loaded area. In the CEB-FIP 1990 model code, at a distance is $2d$. In BS 8110-97, it is $1.5d$, but the peripheral has square corners as compared with CEB-FIP rounded corners.

In North American codes such as ACI 318 and CSA Canadian the control punching shear peripheral is only $0.5d$ away from the loaded column. Different European and North American codes and design guidelines allow the use of shear reinforcement for two-way slab plates. More recently, the American and the Canadian codes allows the use of shear studs developed by Ami Ghali and walter Dilger at Calgary University (1981). Also Punching shear reinforcement has become one of the approved methods in most codes of the Middle East countries such as Egyptian code of practice ECP 203(2017). Corley and Hawkins (1968) suggest details of a shear heads. This mechanism makes use of structural steel sections welded collectively to make a grid which can then be placed around or via a column. Their study shaped the basis of the shear head reinforcement design guidelines and recommendations in the American Code Institute design code ACI 318. The current research develops a new formula to predict contribution of shear heads in resisting punching shear of flat plates.

CODE REQUIREMENTS FOR TWO-WAY SHEAR

Two-way shear design of flat slabs uses the critical sections (control perimeter) approach in most design code requirements. The nominal shear strength is determined at a critical section around the column or loaded area. Concrete strength and geometric parameters are usually governing the function of shear strength.

ACI 318-14 CODE PROVISIONS

ACI 318-14 requires the factored shear stress v_c at the critical section (the perimeter at a distance $\frac{d}{2}$ from column faces).

The factored shear resistance of the critical section is the smallest of the values obtained from eqs. [1] up to [3]:

$$v_c = 0.33 \phi \sqrt{f'_c} \quad (1)$$

$$v_c = 0.083 \phi \sqrt{f'_c} \left(2 + \frac{4}{\beta_c} \right) \quad (2)$$

$$v_c = 0.083 \phi \sqrt{f'_c} \left(2 + \frac{\alpha_s d}{b_o} \right) \quad (3)$$

Where β_c is the ratio of the long side over short side of the column, b_o is the perimeter length of the critical section. $\alpha_s = 40, 30$ and 20 for interior, edge, and corner column, respectively.

EURO CODE 2 PROVISIONS

The Euro code 2 (2004) displays a basic simple control section at a distance $2d$ from the faces of the column or the loaded area. Further, the shear stress v_f on the control section should be no more than the shear resistance (V_r).

$$V_f \leq V_r \quad (4)$$

As shown in Figure (1), for rectangular columns, the basic control section includes round corners (ACI and CSA code permit right angle corners). The code also requires checks on the column face and on the control section outside the shear reinforcement area. For interior slab-column connections without shear reinforcements, the shear resistance V_r for the basic control section is calculated as:

$$V_c = \frac{0.18}{\gamma_c} k (100 \rho_l f_{ck})^{1/3} 0.10 \sigma_{cp} \geq (V_{min} + 0.10 \sigma_{cp}) \quad (5)$$

$$k = 1 + \left(\frac{200}{d} \right)^{0.5} < 2, d \text{ in mm} \quad (6)$$

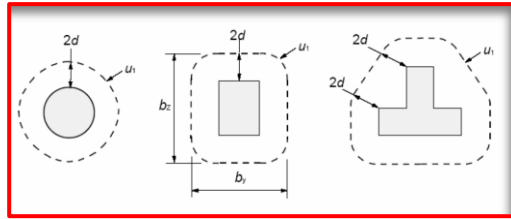


Figure 1: Critical Sections Defined in Euro Code 2 (2004)

f_{ck} = the characteristic concrete strength, MPa

$$\rho_l = (\rho_z \rho_y)^{1/2} \leq 0.02 \quad (7)$$

ρ_z, ρ_y are reinforcing ratios in z, y directions for a slab width equal to column width plus $3d$ each side. $\gamma_c = 1.5$, partial factor for persistent and transient concrete.

$$V_{min} = 0.035 k^{3/2} f_{ck}^{1/2} \quad (8)$$

The shear stress v_f at the basic control section due to factored external concentric load V_f .

$$v_f = \frac{V_f}{u_1 d} \quad (9)$$

For interior columns, the shear stress " v_f " at the column face is

$$v_f = \frac{V_f}{u_0 d} \quad (10)$$

Where u_0 is the length of column perimeter (for interior column)

ECP 203 CODE PROVISIONS

The ECP 203 (2017) employs a basic control critical section at a distance ($d/2$) from the faces of the column.

v_c = punching shear strength of concrete according to () ECP 203-2017

v_c =The least value from the following equations:

$$v_c = 0.316 \left(0.50 + \frac{a}{b} \right) \sqrt{\frac{f_{cu}}{\gamma_c}} \quad (11)$$

$$v_c = 0.316 \sqrt{\frac{f_{cu}}{\gamma_c}} \quad (12)$$

$$v_c = 0.80 \left(\frac{\alpha \cdot d}{b_o} + 0.20 \right) \sqrt{\frac{f_{cu}}{\gamma_c}} \quad \text{but not more than } v_c = 1.7 \text{ N/mm}^2 \quad (13)$$

Where (a/b) is the ratio of the short side over long side of the column, b_o is the perimeter length of the critical section, $\alpha = 4, 3, 2$ for interior, edge, and corner column, respectively. γ_c is reduction factor for compressive strength of concrete.

PUNCHING SHEAR DESIGN OF FLAT SLABS WITH SHEAR HEADS REINFORCEMENT

ACI 318-14 CODE PROVISIONS

ACI 318-14 is the only code which predict the resisting punching force of flat slabs with shear heads, to increase shear strength by steel (I) shapes (shear heads), the following steps should be followed:

a- Calculate total factored shear force [V_u] according to the following equation:

$$V_u = W_{su} \cdot A \quad (14)$$

where $A = (L \cdot B) - (a \cdot b)$

b- Calculate shear strength (v_c) without shear reinforcement in which (v_c) is the smallest of the values obtained eqs. [1] up to [3].

c- Check the safety by comparing (V_u) with (Φv_c) where $\Phi=0.75$

If the safety is not satisfied, it should be increase the shear strength by steel (I) shapes "Shear heads"

d- Check maximum shear strength permitted with steel shapes

$$V_n = \Phi (0.58 \sqrt{f_c'} b_{o1} d) \quad (15)$$

e- Determine minimum required perimeter (b_{o2}) of a critical section at shear head ends with shear strength limited to

$$V_n = \Phi (0.33 \sqrt{f_c'} b_{o2} d) \quad (16)$$

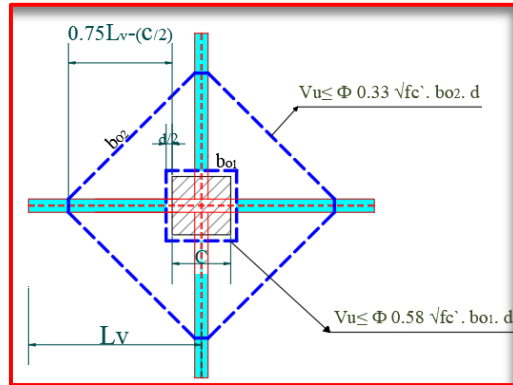


Figure (2): Allowable Punching Shear Strength at Different Perimeters b_{o1} and b_{o2} , f- Determine required length of shear head arm " L_v " to satisfy b_{o2} .

Figure (2) shows allowable punching shear strength at different perimeters b_{o1} and b_{o2} . Strength of slab is reached; the plastic moment required to ensure that the ultimate shear is attained as the moment strength of the shear head is reached should be calculated as follows:

$$\Phi M_p = \frac{V_u}{2n} \left[h_v + \alpha_v \left(L_v - \frac{c}{2} \right) \right] \quad (17)$$

Where:

M_p : plastic moment strength for each shear head arm.

Φ : strength reduction factor for tension controlled member, equal to 0.9.

- n: number of shear head arms.
- L_v : minimum required length of shear head arm.
- h_v : depth of shear head cross-section.
- α_v : the ratio between the flexural stiffness of each shear head arm and that of the surrounding composite cracked slab section of width $(c+d)$ [Assume $\alpha_v=0.25$]
- f- Determine required length of shear head arm " L_v " to satisfy b_{02}
- g- Check depth limitation of shear head web ($h_v \leq 70 t_w$).
- h- Determine location of composite flange of steel shape with respect to compression surface of slab in which compression flange must be located at $0.30d$.

RESEARCH SIGNIFICANCE

The two-way slab system is a significant efficient structural system. It is economical and is widely used in different structural applications such as floors, roofs of buildings and walls of tanks. Shear heads is one of the most important Punching shear reinforcement. Contribution of shear heads in resisting punching shear of flat slabs defined only in ACI 318-14. In an attempt to propose a new design formula to predict punching shear capacity of flat slabs with shear heads. Experimental, numerical and analytical analysis were done taking column aspect ratio into consideration.

EXPERIMENTAL PROGRAM

MATERIALS

The concrete mix was provided by housing and building research center "HBRC" lab.

$$b_{02} = 4\sqrt{2}\left[\frac{c}{2} + \frac{3}{4}\left(L_v - \frac{c}{2}\right)\right] \quad (18)$$

To ensure that premature flexural failure of shear heads doesn't occur before shear

The concrete was designed to have a 28-days cube compressive strength of 25MPa.

Table (1) presents the mix proportions to produce one cubic meter of the concrete mix.

The tensile mechanical properties of the steel bars were tested according to ASTM A370-97a.

Table (2) presents the yield strength, ultimate strength and elongation.

Table (3) presents the yield strength, ultimate strength and elongation of steel in which shear heads made from.

TEST SLABS

A total of nine half-scale flat slab specimens were tested, all tested flat slabs with overall thickness equal to 150 mm and span equal 2000 mm in both directions with clear spans between supported beams equal to 1800 x 1800 mm. A total of nine slabs with square and rectangle column heads, were tested under punching shear loading. The concrete cover used was 10 mm to the bottom face of all test specimens. All slabs were reinforced with bottom longitudinal steel bars mesh $\Phi 18@100\text{mm}$ and top mesh with $\Phi 10@200\text{mm}$. All columns were reinforced with four longitudinal steel bars $\Phi 18$ and confined with $\Phi 10@100\text{mm}$ bars as transverse reinforcements. Table (4) summarizes the general description of the test specimens. Figure (3) shows specimens with steel shear heads during casting. Test setup of all specimens could be showed in Figure (4).

ELECTRICAL STRAIN GAUGES

Four strain gauges were used to monitor the steel reinforcement strain in the two directions at column face as shown in Figure (5). Deflection of specimens are measured using five linear variable differential transducers, at mid-span and two at distance equal $L/8=225\text{-mm}$ and $L/4= 450\text{-mm}$ from the center in each direction as shown in Figure (6) where L is the distance between supports.

Also, Electrical strain gauges with 10 mm length, 119.8 ± 0.2 ohms' resistance were used to measure shear strain in web and longitudinal strains of top and bottom flanges of steel shear head sections at $d/2$ and $0.75(L_v - c/2)$ from column face in an attempt for measure contribution of shear heads in punching shear and also to know however steel flanges reached to yielding or not, Figure (7) and (8) shows typical arrangement and configuration of strain gauges in web and flanges of steel shear sections, then at every stage of loading, radial and tangential cracks were observed and marked.



Figure (3): Specimens with Steel Shear Heads during Casting



Figure (4): Test Setup of Specimens.



Figure (5): Locations of Steel Strain Gauges.



Figure (6): Arrangement of LVDTs over the Specimen.



Figure (7): Electrical Strain Gauges of Steel Shear Heads.

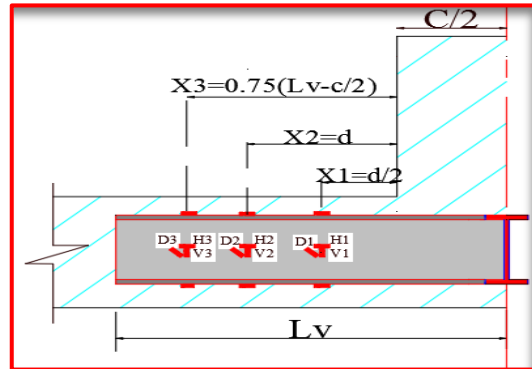


Figure (8): Typical Configuration of Strain Gauges in Web of Shear Heads.

I. TEST RESULTS AND DISCUSSION

This section explains the test results and observed behavior of the tested flat slabs. The experimental results include the crack pattern characteristics, records of punching load capacity versus vertical deflection, strain in tensile reinforcement mesh, strains and contribution of shear head sections.

CRACK PATTERN CHARACTERISTIC

The mode of failure was brittle since the failure occurred in concrete due to punching shear. This doesn't prevent some minor flexural cracks. This can be interpreted as after punching failure of specimen, the specimen returns to bear some flexural stresses which in turn leads to flexural cracks. The crack pattern and mode of failure at the top and bottom of the flat slab specimen (S-L1-2) plotted in Figures (9), (10).

LOAD-VERTICAL DEFLECTION RELATION

The deflections were measured by LVDT gauges at equally spaced locations. The deflection profiles showed the deformational response to the application of load along the slab width. Figures (11), (12) and (13) show the load- vertical deflection curves for specimens with column aspect ratio (1:1), (2:1) and (4:1).



Figure (9): Crack pattern at Tension Zone of Specimen with Shear Heads.



Figure (10): Crack Pattern at Compression Zone of Specimen with Shear Heads.

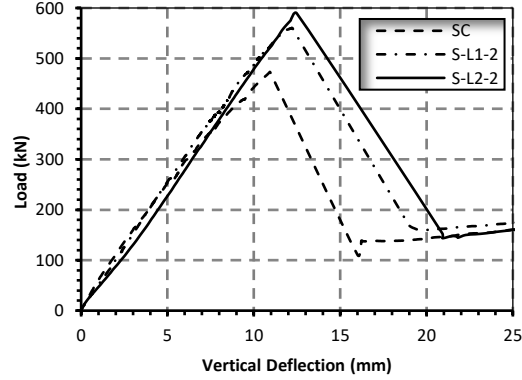


Figure (11): The Load-vertical deflection curve for Specimens with Column Aspect Ratio (1:1)

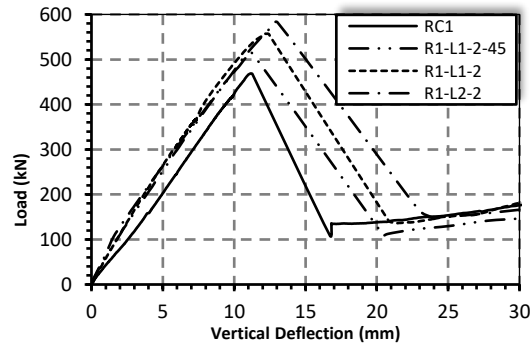


Figure (12): The Load-vertical deflection curve for Specimens with Column Aspect Ratio (2:1)

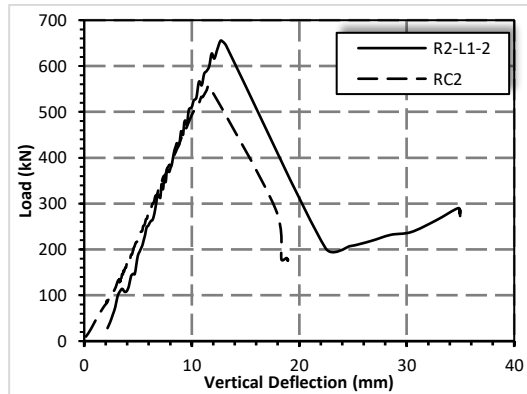


Figure (13): The Load-vertical deflection curve for Specimens with Column Aspect Ratio (4:1)

RECORDED STRAIN IN TENSION STEEL AT COLUMN FACE

The maximum tension steel strain was recorded at column face for all tested specimens. The recorded tension steel strain for specimens with steel shear heads reached yielding point. This recorded bottom steel strain confirms the (punching/flexural) failure occurrence compared with the control specimens. For example,

Figure (14) shows the load versus recorded strain in tension steel at column face for specimens with column aspect ratio (1:1).

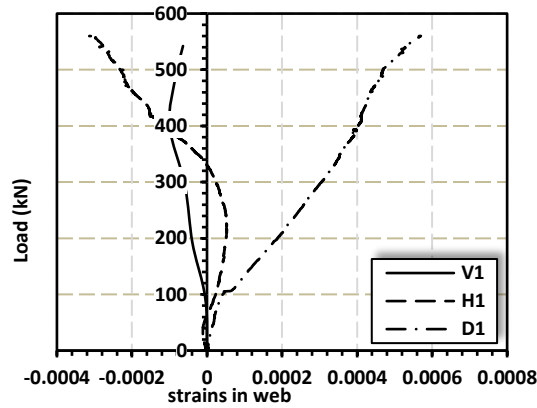


Figure (15): Applied Load versus Strains of Web at $d/2$ for Specimen (S-L1-2).

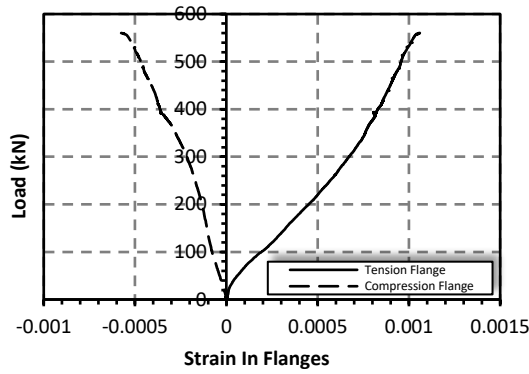


Figure (16): Applied Load versus Strains of Web at $d/2$ for Specimen (S-L1-2).

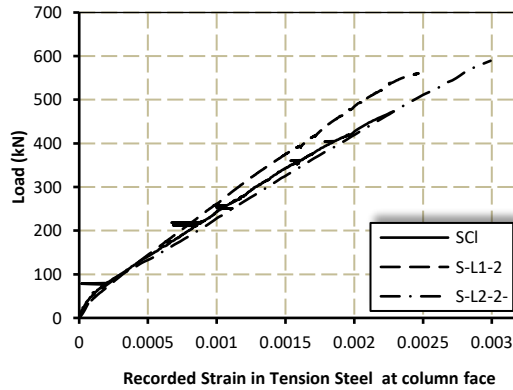


Figure (14): The Load versus Recorded Strain in Tension Steel at Column Face for Specimens with Column Aspect Ratio (1:1).

RECORDED STRAINS AND CONTRIBUTION OF SHEAR HEADS

Recorded normal strains in web, top and bottom flange at distances $d/2$, d and $0.75(L_v - c/2)$ from column face were recorded for all specimens with shear heads. For example, Figures (15) and (16) show results of groups from recorded normal strains at distance $d/2$ for specimen (S-L1-2) in web and flange respectively.

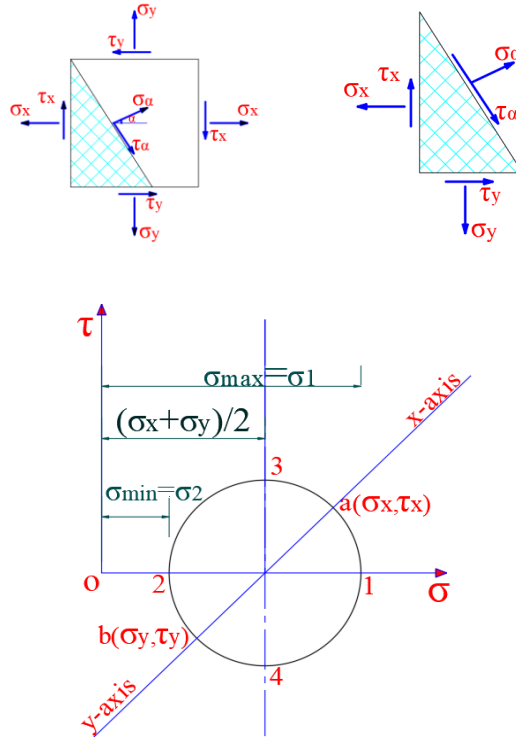


Figure (17): Stress Transformation : Graphical Illustration [Mohr Circle].

The following equations explain the mechanism of calculating contribution of shear heads in punching shear as shown in Figure (17).

$$\sigma_{\alpha} = \frac{\sigma_x + \sigma_y}{2} + \frac{\sigma_x - \sigma_y}{2} \cos 2\alpha - \tau_x \sin 2\alpha \quad (19)$$

$$V = 4\tau_x \cdot A_{web} \quad (20)$$

The number "4" indicated for four leg sides of shear heads. For example, contribution of shear heads for specimens with shear heads in which connected by rectangle column shows in Figure (18).

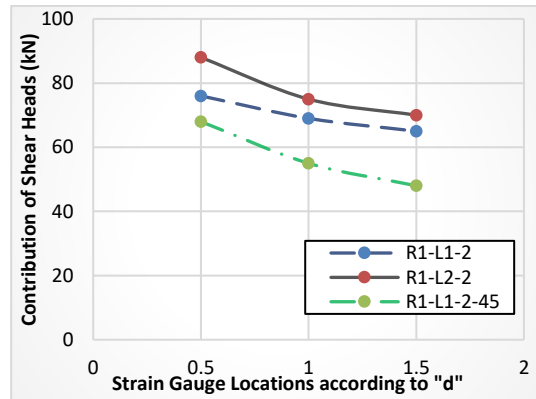


Figure (18): Contribution of Steel Shear Heads for Specimens which Connected by Rectangle Columns.

Two nodes are required for this element. Each node has three degrees of freedom, translation in the nodal x, y, and z direction. The element is also capable of plastic deformation.

SHELL 181, is suitable for analyzing thin to moderate-thick element with six degrees of freedom at each node and valid for simulating steel shear head sections.

SOLID 45 element used for simulate steel plates at the supports for the column. This element has eight nodes with three degrees of freedom.

Finite element modelling for flat slab specimen with shear heads shown in Figure (19).

Material properties for concrete is well described in finite element program. There are multiple parts of the material model for the concrete element as mentioned by Badawy M.M. et. al. for the concrete element, this material model refers to solid 65 elements. And it is defined as linear isotropic for the elastic zone and multilinear isotropic for the plastic zone of the concrete. The steel for the finite element models was assumed to be an elastic-perfectly plastic material and identical in tension and compression. The material modeled as a linear isotropic element with modulus of elasticity for the steel (E_s), and poisson's ratio (PRXY) for lead plates and supports. The material model refers to the SOLID 45 element.

NUMERICAL ANALYSIS PROGRAM

Performing a numerical and analytical study then compared with additional experimental investigation of flat slabs with shear heads to enhance the understanding of the behavior and the mechanism of punching failure.

All details of modelling have been described using finite element program (ANSYS 12.0).

Solid65, an eight-node solid element, is used to model the concrete. The solid element has eight nodes with three degrees of freedom at each node-translations in the nodal x, y, and z directions. The element is capable of plastic deformation, cracking in three orthogonal directions, and crushing.

Link8, truss element is used to model the steel reinforcement.

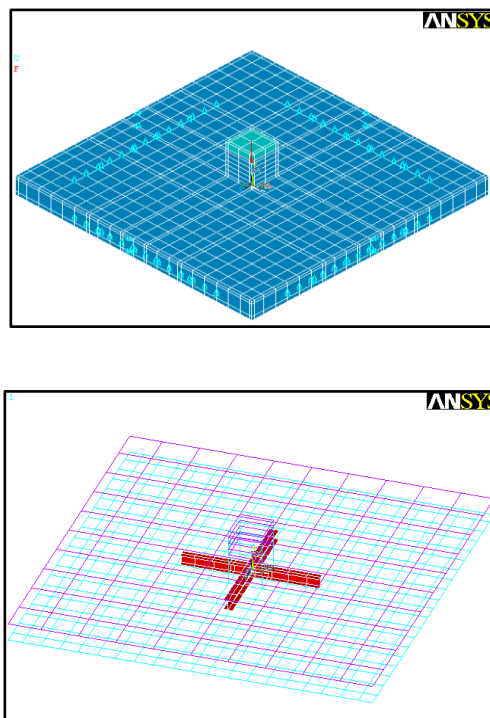


Figure (19): Finite Element Modelling for Flat Slab Specimen with Shear Heads.

Table (1): Concrete Mix Proportions.

Material	Dolomite(kg)	Sand(kg)	Cement(kg)	Water(Litre/kg)
Mix Proportion (Kg/m ³)	898	863	384	230

Table (2): The yield Strength, Ultimate Strength and Elongation.

Material	Nominal Diameter (mm)	Yield Strength (N/mm ²)	Yield Strain	Ultimate Strength (N/mm ²)
Steel (40/60)	10	440	0.002	624
Steel (40/60)	18	490	0.0025	668

Table (3): The yield strength, ultimate strength and elongation of steel in which shear heads made from.

Material	Yield Strength (N/mm ²)	Yield Strain	Ultimate Strength (N/mm ²)
Steel 37	300	0.0015	440

Table (4): The yield strength, ultimate strength and elongation of steel in which shear heads made from.

Group	Specimen	Column Aspect Ratio	Column Dimensions	Shear Head Length	Remarks
Group (A)	SC	1	220*220	without	Control specimen
	S-L1-2	1	220*220	1.75 h	Cut end angle=90°
	S-L2-2	1	220*220	2.25 h	Cut end angle=90°
Group (B)	RC1	2	150*300	without	Control specimen
	R1-L1-2	2	150*300	1.75 h	Cut end angle=90°
	R1-L1-2-45	2	150*300	1.75 h	Cut end angle=45°
	R1-L2-2	2	150*300	2.25 h	Cut end angle=90°
Group(C)	RC2	4	110*440	without	Control specimen
	R2-L1-2	4	110*440	1.75 h	Cut end angle=90°

PARAMETRIC STUDY

A wide range of parametric study to investigate influence of using of embedded steel shear head sections on reinforced concrete flat slabs under punching shear forces. This parametric study suggested to obtain specific guidelines to help structural engineers who deal with this problem. The first investigated parametric study was as follow:

- 1- Different column aspect ratio (a/b).
- 2- Length of steel shear head section from column face (L).
- 3- Different arrangement of steel shear head sections (A).

In order to investigate the effect of the previous parameters on the behavior of specimens, the specimens are classified in the following flow chart, each with a particular parameter. These groups could be described as follows in Figure (20).

Changing compressive strength of concrete (f_{cu}) from 25 MPa to 40 MPa with increment 5 MPa taking in to consideration. This is being the second investigated parametric study which shown in Figure (21). Also another parameter taking into consideration, this parameter was studying the effect of using shear studs welded in shear heads at compression zone (ST). This parameter was made for the specimen which has a column aspect ratio (a/b) equal (1.00) and choice (2.00d) as a length for shear head from column face.

Arrangement (1,1) means using one shear head leg for each direction, Arrangement (2,1) means using two legs of shear heads perpendicular to the long direction of the column, More over one leg of shear head perpendicular to short direction. Arrangement (2,0) indicated for using only two shear head legs perpendicular to the long direction. Figures [22-25] show modeling and shear stresses of shear heads for specimens with different arrangement (2,1) and (2,0).

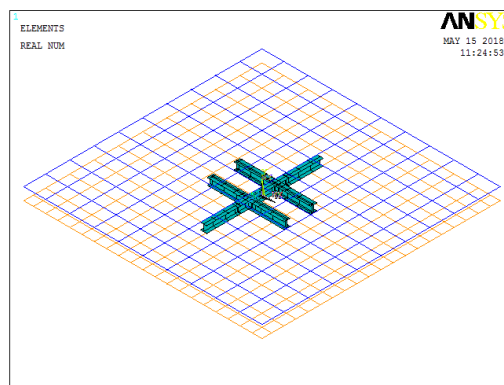


Figure (22): Finite Element Modelling of Specimen R4:1-2.00d-2,1

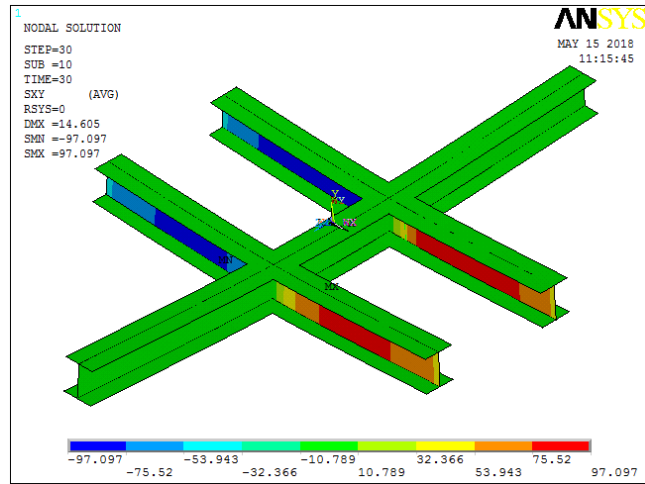


Figure (23): Shear Stresses in Short Legs for Specimen R4:1-2.00d-2,1

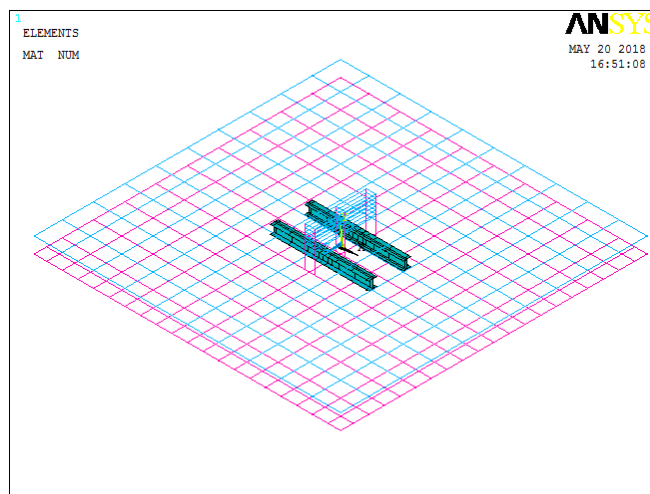


Figure (24): Finite Element Modelling of Specimen R4:1-2.00d-2,0

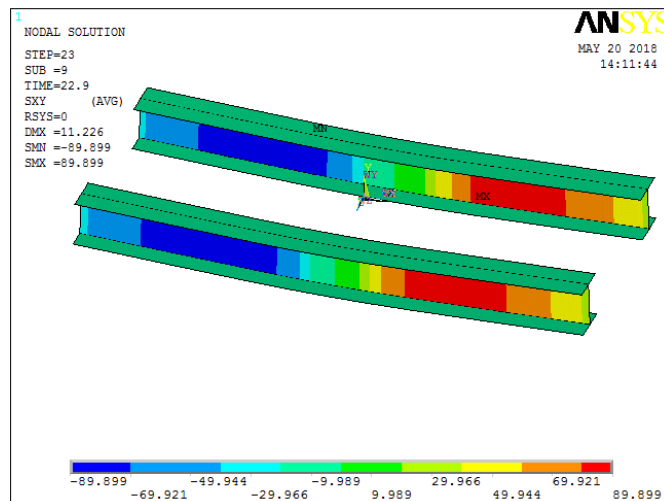


Figure (25): Shear Stresses in Legs for Specimen R4:1-2.00d-2,0

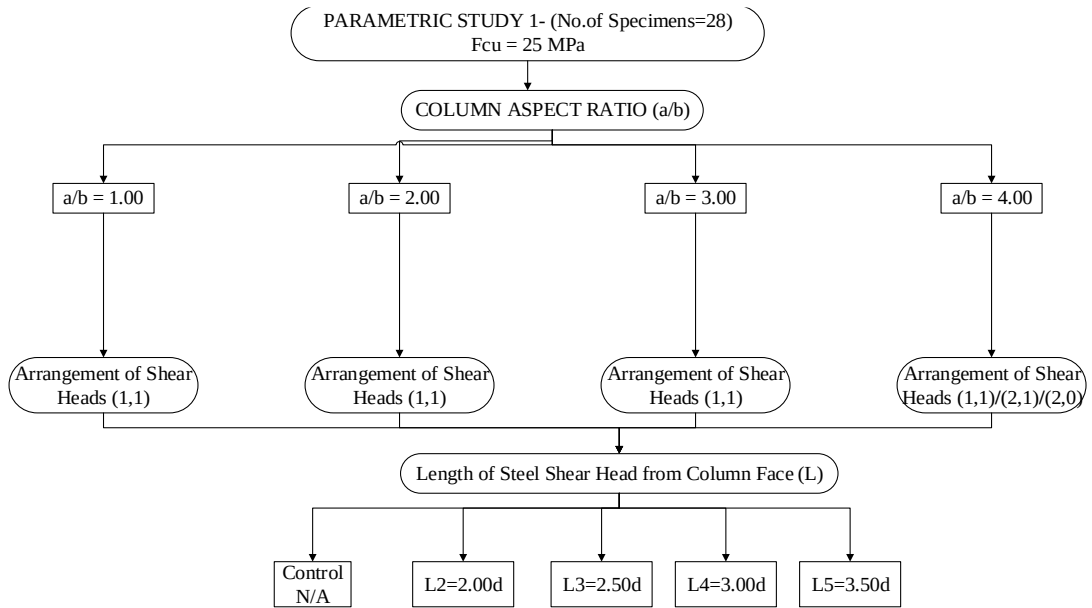


Figure (20): Parametric Study (I).

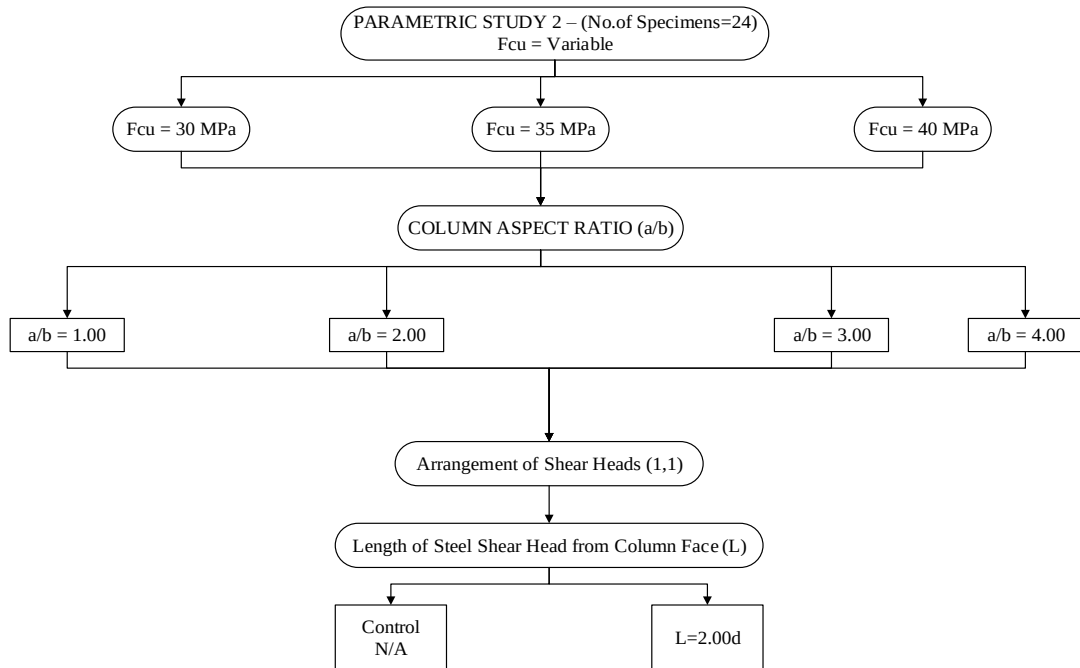


Figure (21): Parametric Study (II).

Figure (26) shows comparison for specimens of column aspect ratio (4:1) with different arrangement [1,1-2,1-2,0]. A group of new specimens with different main longitudinal steel reinforcement had been made. Main longitudinal steel reinforcement changed to be $\Phi 22@100\text{mm}$ ($\rho=2.5$) instead of $\Phi 18@100\text{mm}$ ($\rho=1.7$). Eight specimens were analyzed using "ANSYS" program. Four different column aspect ratio were used (1:1, 2:1, 3:1 & 4:1) with two different lengths of shear heads (2.00d, 2.5d) from column face. Figure (27) shows results of specimens with different main longitudinal steel reinforcement ratio [$\rho = 1.7\%$ & $\rho = 2.5\%$].

From the observation of the crack pattern at the top of all most specimens. It was found that the cracks began almost beyond the column face. So the cracks created a pathway in the concrete until reaching the beginning of shear heads. From here came the idea of making composite shear reinforcement system between shear heads and studs. Shear studs were welded with shear heads at compression zone of the slab. This is in order to try to increase the punching circumference at compression zone of the slab. Specimens with column

aspect ratio (1:1) were chosen for this study and have the same compressive strength of concrete (25MPa). This comparison has been made for different lengths of shear heads which ranged from (2.00d) to (3.50d).

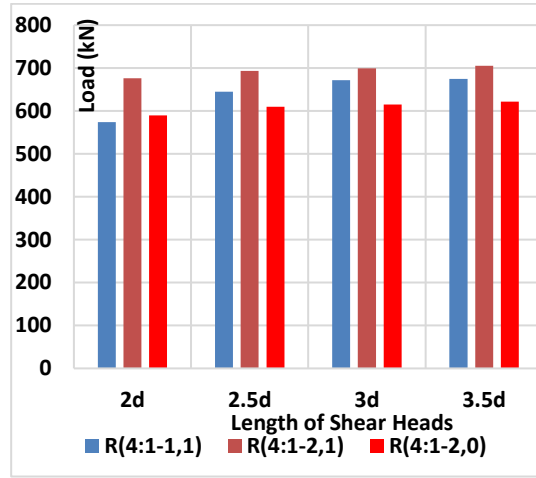


Figure (26): Comparison for Specimens of Column Aspect Ratio (4:1) with Different Arrangement [1,1-2,1-2,0].

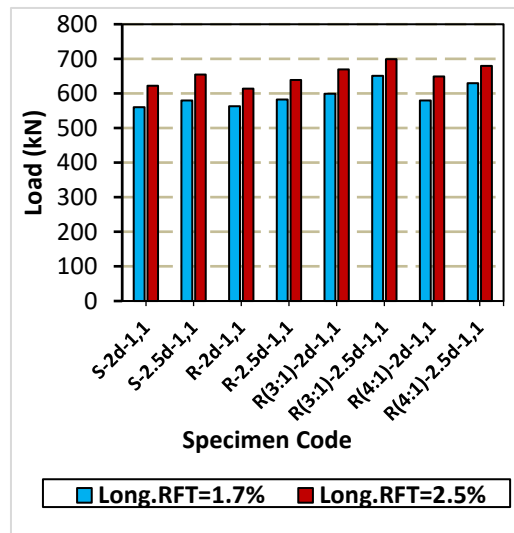


Figure (27): Bar Chart of Specimens with Different Main Longitudinal Steel Reinforcement Ratio [$\rho = 1.7\%$ & $\rho = 2.5\%$]

PROPOSED EQUATION

Figure (28) shows philosophy of crack path which was derived from experimental program. The proposed equation was constructed on two perimeters, small perimeter at compression zone and large perimeter at tension zone.

These steps could be followed to obtain the parameters in the proposed equation:

$$b_{o1} = 2[(a + a_1) + (b + a_1)] \quad (19)$$

$$a_1 + a_2 = d \quad (20)$$

By regression analysis:

$$A = a + 0.48 \times d \quad (21)$$

$$B = b + 1.45d \quad (22)$$

$$L_x = L_{sh(x)} + \frac{a_2}{2} \quad (23)$$

$$L_y = L_{sh(y)} + \frac{a_2}{2} \quad (24)$$

$$L' = \sqrt{\left(L_x - \frac{A}{2}\right)^2 + \left(L_y - \frac{B}{2}\right)^2} \quad (25)$$

$$b_{o2} = 4L' + 2A + 2B \quad (26)$$

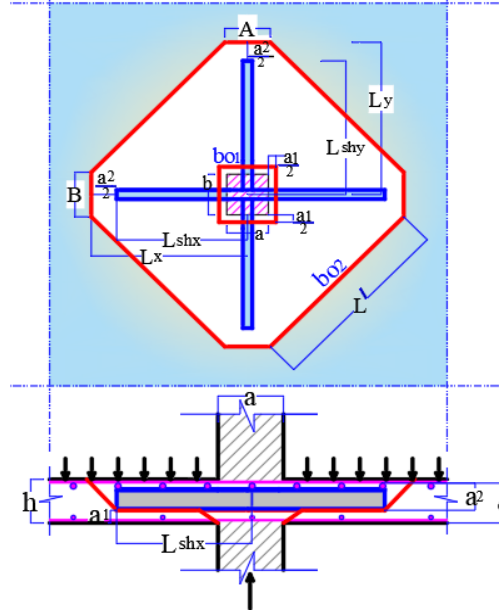


Figure (28): Crack Path Model for Flat Slabs with Shear Heads.

a, b are short and long sides of column respectively.

The proposed design formula for punching load capacity could be expressed as follows

$$V = 0.63 \left(0.5 + \frac{a}{b}\right) \sqrt{f_{cu}} \times b_{o1} \times a_1 + 0.26 \times \sqrt{f_{cu}} \times b_{o2} \times a_2 \quad (27)$$

Where:

$$0.5 + \frac{a}{b} \leq 1.00$$

V: Ultimate Punching Load (N).

f_{cu} : Standard Cubic Strength of Concrete (MPa).

b_{o1} : Punching Perimeter at Compression Zone (mm).

a_1 : Distance from Compression Fiber of Concrete up to Compression Fiber of Shear Heads (mm).

b_{o2} : Punching Perimeter at Tension Zone (mm).

a_2 : Distance from Compression Fiber of Shear Heads up to c.g of Tension Reinforcement.

Figure (29) shows comparison between experimental, numerical and proposed design equation. This proposed design equation is conservative because of all expected results from proposed equation are less than the results of the experimental and numerical analysis. This is due to the brittle failure of flat slabs under punching shear.

Figure (30) shows crack path model where suggested by Sherif and Dilger (1995). Failure perimeter is proposed outside of shear reinforcement zone at distance $d/2$ from shear head ends.

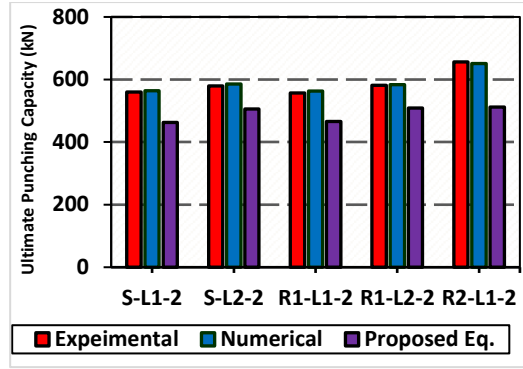


Figure (29): Comparison between Experimental, Numerical and Proposed Design Equations.

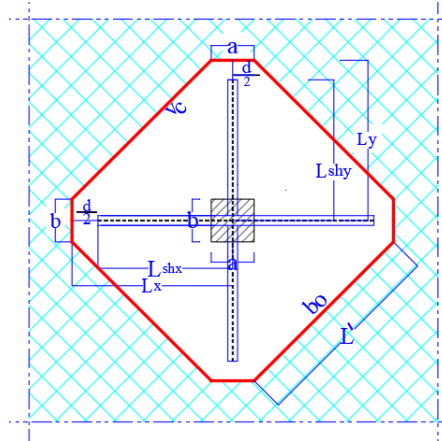


Figure (30): Crack Path Model Suggested by Sherif and Dilger (1995).

COMPARISON USING "SHERIF AND DILGER" (1995) FORMULA

Sherif and Dilger (1995) recommended that the shear stress resistance be expressed as a function of the distance from column face and percentage of main longitudinal reinforcement.

$$v_c = 0.7(100\rho f'_c)^{\frac{1}{3}} \left(0.1 + \frac{1}{\alpha_o^{0.63}}\right) \leq 0.7(100\rho f'_c)^{\frac{1}{3}} \quad (28)$$

Where:

v_c : punching shear stress resistance at a critical section.

ρ : percentage of main longitudinal reinforcement.

α_o : Distance of shear head from column face divided by depth.

f'_c : Cylindrical compressive strength of concrete.

Because of the decrease of shear stress resistance below $0.2\sqrt{f'_c}$ (value specified by CSA) for $\alpha_o > 5$, it is recommended that α_o should not exceed 5, which means that shear reinforcement should not be extended beyond a distance of $4.5d$.

Figure (31) shows shear stress resistance as a function of distance from column face divided by depth "d".

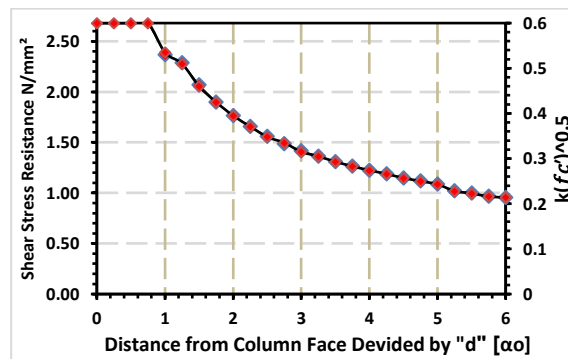


Figure (31): Shear Stress Resistance as a function of distance from column face divided by depth "d".

CONCLUSION

Flat slabs with shear heads enhanced the mode of failure to be semi brittle failure. The European code (EC2-2004) is the closest code for finding concrete contribution in punching shear resistance. The new proposed formula has an original contribution and a good agreement in prediction of punching shear capacity of flat slabs with shear heads. Shear heads with length equal to $2d$, $2.5d$, $3d$, $3.5d$ enhanced punching shear capacity of flat slabs by 18, 22, 33 and 31.6 respectively for flat slabs with column aspect ratio 1:1, 2:1, 3:1 and 4:1. Punching shear capacity of flat slabs enhanced by 10, 19 and 27% when compressive strength of concrete changed to be 30, 35 and 40 MPa respectively.

Rates of improvement in punching shear force for specimens with the new arrangement (2,1) higher by (16.5, 10, 5 and 4%) compared to specimens with arrangement (1,1) at the same used lengths of shear heads ($2d$, $2.5d$, $3d$ and $3.5d$) respectively. The new arrangement of shear heads (2,0) showed its inability to increase the punching shear resistance of flat slabs with column aspect ratio (4:1) compared to the basic arrangement (1,1) at the same used lengths of shear heads ($2d$, $2.5d$, $3d$ and $3.5d$). To avoid slippage of steel shear heads in flat slabs, Minimum length of ($2.00d$) from column face is the preferred length for shear heads. Maximum optimum length of shear heads for flat slabs with column aspect ratio (1:1), (2:1), (3:1) and (4:1) is equal to ($3.00d$) from column face.

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Influence of cohesive soil depth on compressibility coefficient and coefficient of volume change

M.A. Mahmoud^{1,3}, A.M. Radwan¹, M.A. Ali², A.A. Farag³

¹Civil Engineering Department, Faculty of Engineering, Helwan University, Egypt

²MAS Consulting Engineering, Benha, Egypt,

³Civil Engineering Department, High Institute for Engineering and Technology Al-Obour, Egypt

Corresponding Author Email:

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Abstract – The settlement is the most serious problem of fine-grained soils which caused by a phenomenon called soil consolidation. Most of previous studies were concerned with studying consolidation conditions depending on the consolidation's theories as Terzaghi's theory. These studies, in some cases, do not give a simulation of reality representation. It was necessary to study the effect of soil depth from ground surface on consolidation parameters to simulate what happens to the fine soil in nature. Therefore, the consolidation behavior of four cohesive soils was investigated in this paper. The studied soil samples were collected from different depths of cohesive soil layers at four different sites, in El-Qalubia governorate, Egypt. A series of laboratory consolidation tests were carried out using Oedometer apparatus. So, the effect of soil sample depths (D) from ground surface on compressibility coefficient (a_v) and volume change coefficient (m_v) was checked. Also, the empirical equations were conducted according to the relationship between soil sample depths D vs. a_v and D vs. m_v .

Keywords – Fine-grained soil, cohesive soil, soil sample depth, consolidation, coefficient of compressibility, coefficient of volume change, settlement

I. INTRODUCTION

Soil layer consolidation means water out between the soil particles when it is influenced by loads or stresses such as: foundations, above constructions and underground constructions loads. The seriousness of soil consolidation appears in the soil layers settlements. This settlement of soil layers leads to many problems of the foundations and structures [1][2][3][4], especially when there are differential settlements [5][6][7]. Moreover, the tendencies can occur in buildings and constructions. Where, fractures, partial collapses and total collapses can occur in structures, especially in cases of high loads [1][7][8][9].

The researchers have exerted great efforts to investigate the consolidation parameters of the cohesive soil layers. That is to estimate realistic and appropriate values of the soil layer compressibility and settlement when it is exposed to stresses resulting from the implementation of foundations and structures. Most applied theories are interested in studying the consolidation parameters of the soil layer to give a simulated representation of the entire layer. It is necessary to study the change in the consolidation parameters with the variant depths of the cohesive layer.

In this research, the changes of consolidation parameters (a_v - m_v) with increasing the depth of cohesive soil layer are investigated and studied. Laboratory tests are performed to determine the consolidation parameters on natural cohesive samples obtained from cohesive soil layers at different depths.

II. EXPERIMENTAL APPLICATION

For experimental applications, the physical properties (index properties) of the studied cohesive soil samples such as natural water content (w_{nat}), natural unit weight (γ_{nat}), liquid limit (LL), plastic limit (PL), shrinkage limit (SL), specific gravity (G_s), free swell (FS), and particle size distribution (sedimentation analysis test) were determined according to Egyptian code (2017). Then, by using unified soil classification system (USCS), a series of consolidation tests were carried out as two-dimensional ones, these tests were performed using oedometer apparatus.

2.1 The Studied Soils

The three studied samples of cohesive soils were obtained from each site at variant depths, since the collected soil samples are located under the ground water table in each site, since the studied cohesive soil samples at the variant depths are collected using mechanical boring as follows:

- The three soil samples (D1, D2 and D3) from cohesive soil layer at site one (S1) were obtained at variant depths 4.00m, 7.00m and 10.00m respectively.
- The three soil samples (D1, D2 and D3) from cohesive soil layer at site two (S2) were obtained at variant depths 7.00m, 10.00 and 13.00m respectively.
- The three soil samples (D1, D2 and D3) from cohesive soil layer at site three (S3) were obtained at variant depths 3.00m, 6.00m and 9.00m respectively.
- The three soil samples (D1, D2 and D3) from cohesive soil layer at site four (S4) were obtained at variant depths 5.00m, 8.00m and 11.00m respectively.
- . Tables 1 to 4 contains the engineering properties of studied soils.

Table 1: Physical properties of the studied soil samples for site 1.

Property	Soil sample No.		
	D1= 4.00m	D2= 7.00m	D3=10.00 m
Natural density (γ_b) t/m ³	1.98	2.01	2.08
Natural Water content (W_c %)	34	33	30
Liquid limit (L.L%)	63	61	58
Plastic limit (PL%)	24	27	25
Plasticity index (PI%)	39	34	33
Shrinkage limit (SL%)	18	16	19
Specific gravity (GS)	2.72	2.74	2.73
Free swell (FS%)	80	75	70
Clay content (%)	41	43	40
Silt content (%)	54	53	54
Fine sand content (%)	5	4	6
Soil classification	CH	CH	CH

Table 2: Physical properties of the studied soil samples for site 2.

Property	Soil sample No.		
	D1= 7.00	D2= 10.00m	D3= 13.00m
Natural density (γ_b) t/m ³	2.11	2.15	2.17
Natural Water content (W_c %)	36	34	33
Liquid limit (L.L%)	78	80	81
Plastic limit (PL%)	24	25	26
Plasticity index (PI%)	54	55	55
Shrinkage limit (SL%)	16	14	13
Specific gravity (GS)	2.75	2.74	2.75
Free swell (FS%)	125	150	125
Clay content (%)	57	60	61
Silt content (%)	43	46	39
Fine sand content (%)	—	—	—
Soil classification	CH	CH	CH

Table 3: Physical properties of the studied soil samples for site 3.

Property	Soil sample No.		
	D1= 3.00m	D2= 6.00m	D3= 9.00m
Natural density (γ_b) t/m ³	1.95	2.00	2.01
Natural Water content (W_c %)	33	32	30
Liquid limit (L.L%)	74	73	70
Plastic limit (PL%)	25	26	24
Plasticity index (PI%)	49	47	46
Shrinkage limit (SL%)	15	17	14
Specific gravity (GS)	2.71	2.73	2.71
Free swell (FS%)	100	125	125
Clay content (%)	52	54	49
Silt content (%)	46	46	49
Fine sand content (%)	2	—	—
Soil classification	CH	CH	CH

Table 4: Physical properties of the studied soil samples for site 4.

Property	Soil sample No.		
	D1= 5.00m	D2=8.00 m	D3= 11.00m
Natural density (γ_b) t/m ³	1.93	1.98	2.00
Natural Water content (W_c %)	37	34	33
Liquid limit (L.L%)	54	50	52
Plastic limit (PL%)	28	26	27
Plasticity index (PI%)	26	24	25
Shrinkage limit (SL%)	20	19	21
Specific gravity (GS)	2.73	2.74	2.72
Free swell (FS%)	60	70	65
Clay content (%)	40	39	41
Silt content (%)	54	51	54
Fine sand content (%)	6	10	5
Soil classification	CH	CH	CH

2.2 Consolidation Apparatus and Testing

The ring from stainless steel are of 20 mm height and 50 mm diameter. The undisturbed natural soil sample was placed inside this ring. Then, the porous stones with 5 mm thickness were placed on top and bottom sides of the studied undisturbed soil sample.

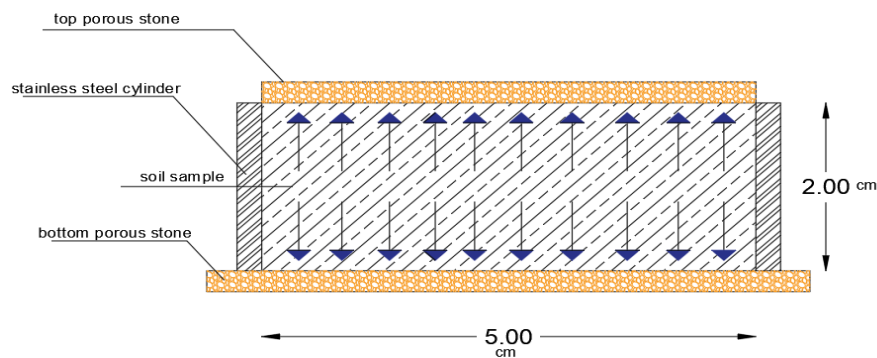


Fig. 1: Model of consolidated soil sample



Fig. 2: Preparation of soil samples for consolidation testing.

Three odometers apparatus were used for laboratory consolidation tests, Fig.3 The consolidation test procedures and reading results were recorded as the following steps:

- The undisturbed cohesive soil sample was enclosed inside the consolidation ring. The weight of the ring must be known in order to measure the soil sample weight. Accordingly, the unit weight (γ_{nat}) of the studied soil can be estimated.
- The used porous stones were immersed in distilled water for 8 hours. Then, the porous stones and filter paper were placed at the top and the bottom sides of drainage test as indicated.
- The loading bad was placed at the top of porous stone and the whole assembly was mounted on the loading frame, since the applied load should be axial and should be centered with soil sample.
- The dial gauge, with accuracy of 0.01 mm/division was arranged in apposition to record the behavior of soil samples in swelling and compressibility. Then, the applied stress by about 0.02 kg/cm² was used to support and to start the consolidation test procedures.
- After completely compressed, the soil sample under a first stress of 0.2546 kg/cm² was applied and stop watch started immediately. The readings of dial gauge were recorded at various time intervals, since the readings were taken at 0.5, 1, 2, 4, 8, 15, 30 minutes, 1, 2, 4, 8, 12, 24 hours till the reading was completely constant. The same procedure was repeated similarly at, 0.2546, 0.509, 1.018, 2.037, 4.074, 8.148 kg/cm².



Fig.3. odometer apparatus

III. RESULTS AND ANALYSIS

Compressibility coefficient (a_v) and Volume change coefficient (m_v) were determined according to consolidation test data of the studied soil samples. These coefficients a_v and m_v were determined using general manner of consolidation theory. So, the relation between compressibility coefficient (a_v) and applied stress increments ($\Delta\sigma$) are shown in Figure 4 for the studied soil samples S1, S2, S3 and S4 respectively, while Fig. 5 shows the relationship between volume change coefficient (m_v) and each applied stress ($\Delta\sigma$) for the studied soil samples S1, S2, S3 and S4 respectively.

3.1 Coefficient of Compressibility (a_v)

The obtained values of compressibility coefficient (a_v) which were determined for the studied cohesive soil samples with different depths of cohesive soil layer at variant sites (S1 to S4) are indicated in Figure 4, since this figure represents the relationships between coefficient of compressibility (a_v) and applied stress increment ($\Delta\sigma$). In the other side, table 5 contains the comparison between a_v values of the studied soil samples

for different depths at variant sites at ($\Delta\sigma=1.018 \text{ kg/cm}^2$) on the studied cohesive soil samples at variant sites (S1 to S4) respectively.

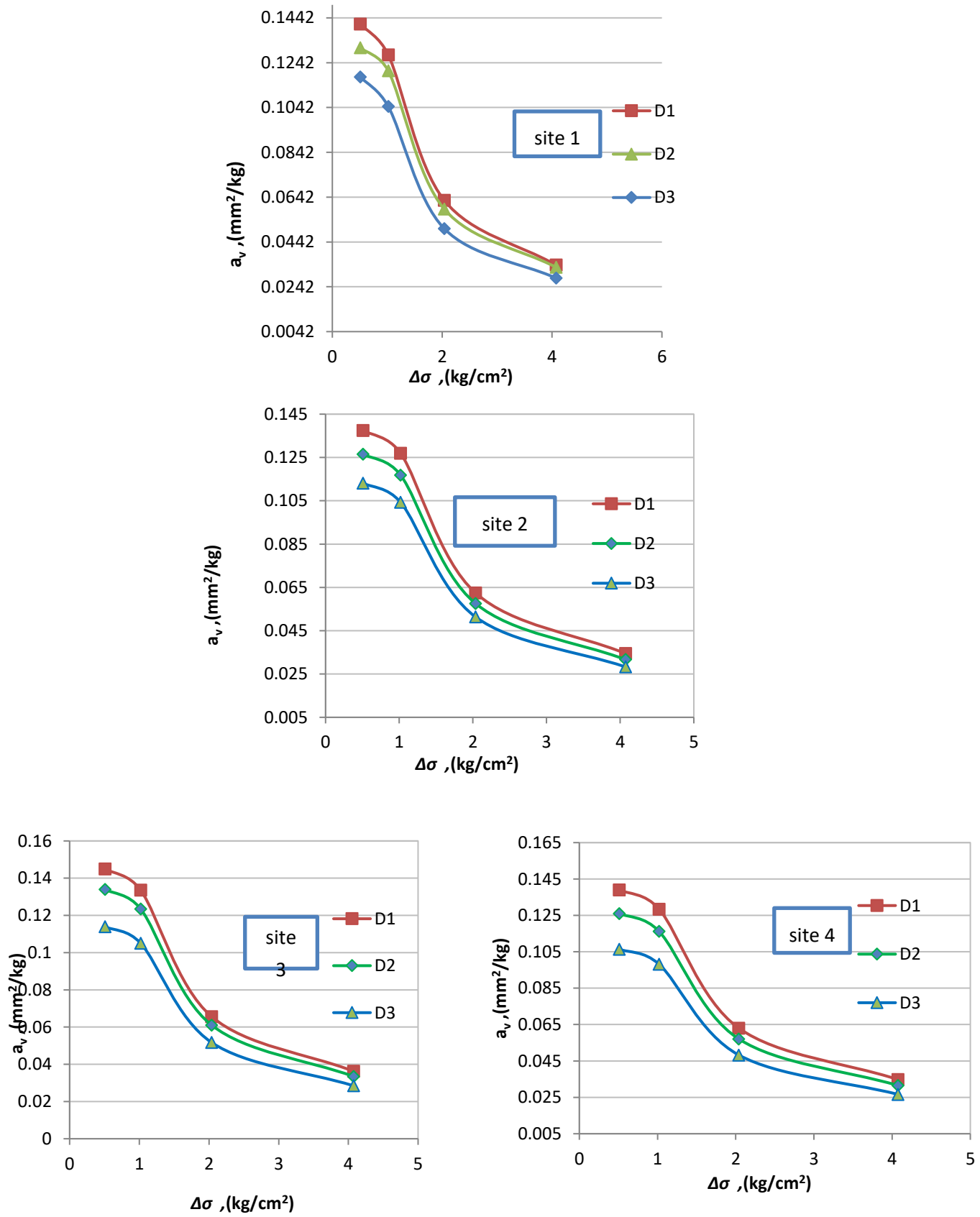


Fig. 4: Relation between a_v and $\Delta\sigma$ for different soil samples depths at sites.

Table 5: Comparison a_v values of soil samples for different depths at variant sites

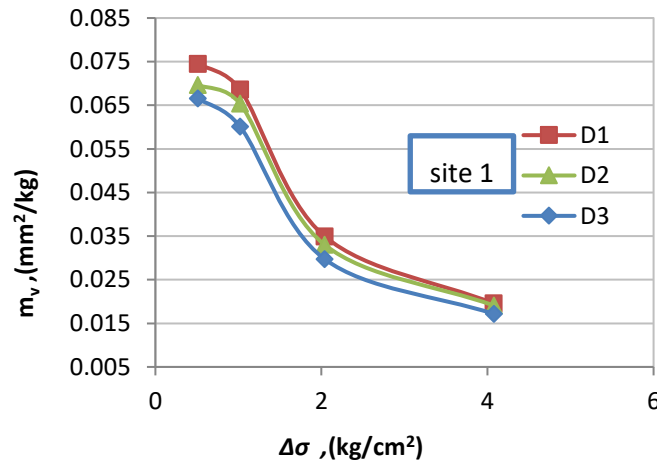
Site	a_v (mm ² /kg) at $\Delta\sigma = 1.018$ kg/cm ²			Decrease percentage at D3	
	D1	D2	D3	D1	D2
S1	0.06281	0.0589	0.05017	20.12%	14.82%
S2	0.0624	0.05753	0.05135	17.71%	10.75%
S3	0.0657	0.0611	0.0517	21.31%	15.38%
S4	0.0631	0.0571	0.0483	23.45%	15.41%
Average decrease percentage				20.29%	13.72%

Referring to figure 4 and table 5 about the estimated values of the compressibility coefficient (a_v), it is noticed that:

- The values of a_v decrease gradually with the increase of applied pressure.
- The values of a_v are related to the change of voids ratio (Δe) and the complete compressibility of soil ($U=100\%$).
- The a_v values of consolidated soil are depending on the studied cohesive soil sample depth from ground surface.
- The values of a_v are decreased with the increase of the soil sample depth under applied stress increment ($\Delta\sigma$). For example, at $\Delta\sigma=1.018$ kg/cm²:
 - For site one (S1), the values of a_v for D3 are decreased by about 20.12% and 14.82% than that for D1 and D2 respectively.
 - For site two (S2), the values of a_v for D3 are decreased by about 17.71% and 10.75% than that for D1 and D2 respectively.
 - For site three (S3), the values of a_v for D3 are decreased by about 21.31% and 15.38% than that for D1 and D2 respectively.
 - For site four (S4), the values of a_v for D3 are decreased by about 23.45% and 15.41% than that for D1 and D2 respectively.

3.2 Coefficient Volume Change (m_v)

The obtained values of volume change coefficient (m_v) which were determined for the studied cohesive soil samples with different depths of cohesive soil layer at variant sites (S1 to S4) are indicated in Figure 5, since this figure represents the relationships between volume change coefficient (m_v) and applied stress increment ($\Delta\sigma$). In the other side, table 6 contains the comparison between m_v values of the studied soil samples for different depths at variant sites at ($\Delta\sigma=1.018$ kg/cm²) on the studied cohesive soil samples at variant sites (S1 to S4) respectively.



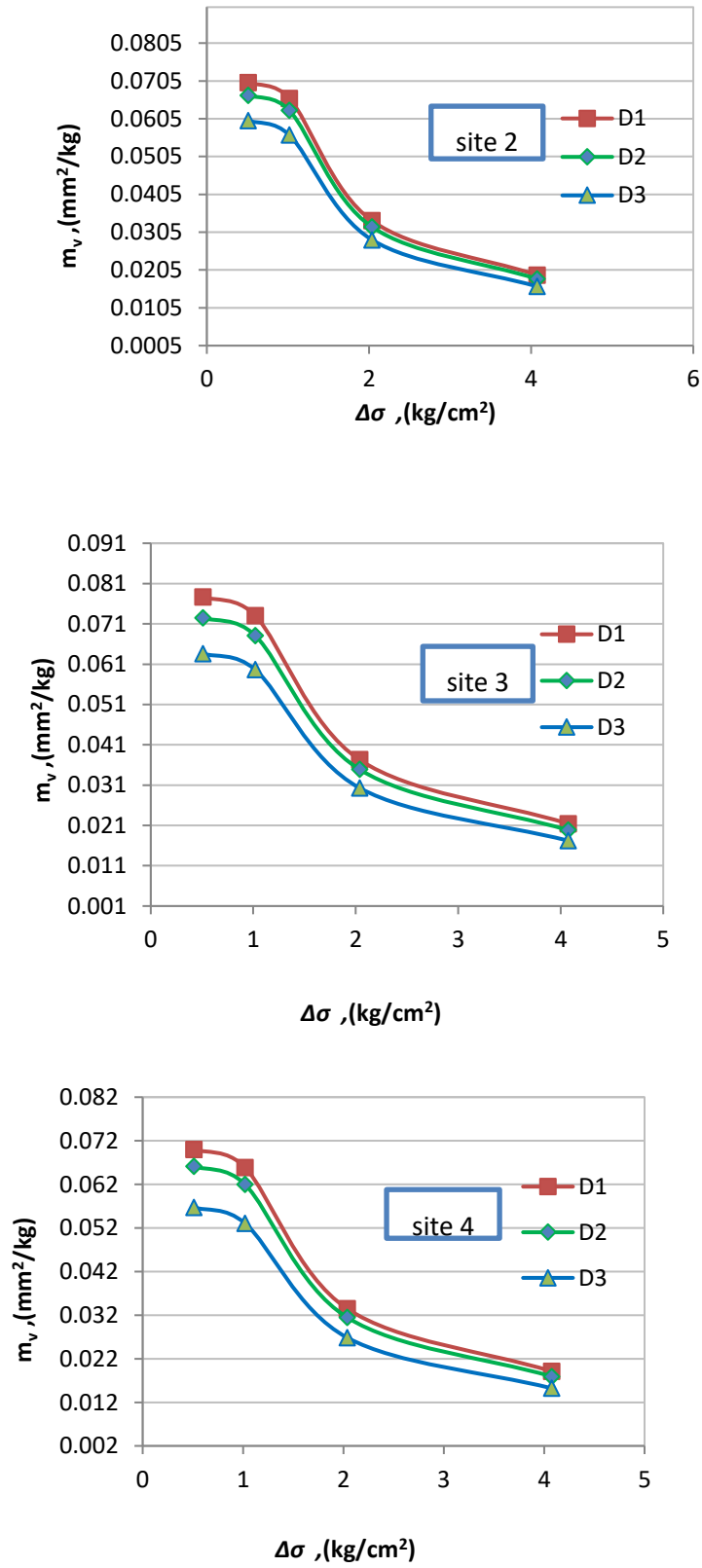


Fig. 5: Relation between m_v and $\Delta\sigma$ for different soil samples depths at sites.

Table 6: Comparison among m_v Values of Soil Samples for Different Depths at variant sites.

Site	m_v (mm ² /kg) at $\Delta\sigma = 1.018$ kg/cm ²			Decrease percentage at D3	
	D1	D2	D3	D1	D2
S1	0.0349	0.03304	0.02974	14.79%	9.99%
S2	0.03352	0.03188	0.02848	15.04%	10.67%
S3	0.0373	0.0349	0.0303	18.77%	13.18%
S4	0.03347	0.0315	0.0268	19.93%	14.92%
Average decrease percentage				17.13%	12.19%

Referring to figure 5 and table 6 about the estimated values of the volume change coefficient (m_v), it is noticed that:

- The values of m_v decrease gradually with the increase of applied pressure.
- The values of m_v are related to the change of voids ratio (Δe) and the complete compressibility of soil ($U=100\%$).
- The a_v values of consolidated soil depend on the studied cohesive soil sample depth from ground surface.
- The values of m_v are decreased with the increase of the soil sample depth Under applied stress increment ($\Delta\sigma$). For example, at $\Delta\sigma=1.018$ kg/cm²:
 - For site one (S1), the values of m_v for D3 are less than those values for D1 and D2 by about 14.79% and 9.99% respectively.
 - For site two (S2), the values of m_v for D3 are less than those values for D1 and D2 by about 15.04% and 10.67% respectively.
 - For site three (S3), the values of m_v for D3 are less than those values for D1 and D2 by about 18.77% and 13.18% respectively.
 - For site four (S4), the values of m_v for D3 are less than those values for D1 and D2 by about 19.93% and 14.92% respectively.

3.3 Correlation between compressibility coefficient (a_v) with the different depths (D) of the studied cohesive soil samples.

Due to the aforementioned results, the relationships between compressibility coefficient (a_v) and the different depths of the studied cohesive soil samples at applied stress increment ($\Delta\sigma=1.018$ kg/cm²) are shown in Fig.7. So, by using Data-Fit software to correlate the relation between a_v and D, an empirical equation can be deduced for the decreased value of a_v with the increase of D, as the following:

$$\Delta a_v = a_v (-A * D + B)$$

Since:

a_v is the compressibility coefficient (mm²/kg)

Δa_v is the decreased value of compressibility coefficient (mm²/kg)

D is the cohesive soil sample depth from ground surface

A&B are constant numbers

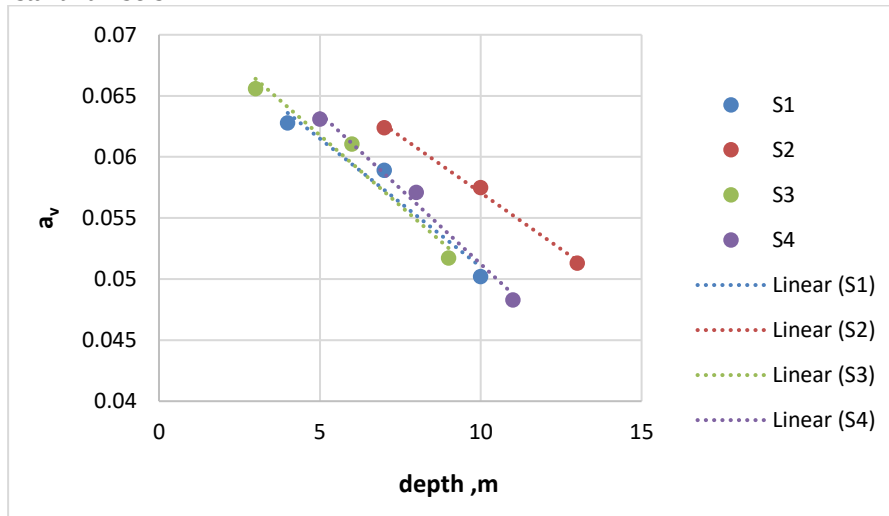


Fig.7: Relation between a_v and D at variant sites

Accordingly, A and B values due to the fitting of the relations between a_v and D, it can be noted that:

A. At site 1, the fitting data can be as:

$$\Delta a_v = a_v * (-0.0021 * D + 0.072)$$

B. At site 2, the fitting data can be as:

$$\Delta a_v = a_v * (-0.0019 D + 0.0756)$$

C. At site 3, the fitting data can be as:

$$\Delta a_v = a_v * (-0.0023 D + 0.0733)$$

D. At site 4, the fitting data can be as;

$$\Delta a_v = a_v * (-0.0025 D + 0.0759)$$

In general, the value of constant A and B for the studied cohesive soil layers at variant sites are about (-0.0019 to -0.0025) and (0.072 to 0.0759) respectively. So, the empirical equations can be applied as:

$$\Delta a_v = a_v * [-(0.0019 - 0.0025) D + (0.072 - 0.0759)]$$

3.4 Correlation between volume change coefficient (m_v) with the different depths (D) of the studied cohesive soil samples.

Due to the aforementioned results, the relationships between volume change coefficient (m_v) and the different depths of the studied cohesive soil samples at applied stress increment ($\Delta\sigma=1.018 \text{ kg/cm}^2$) are shown in Fig. 8. So, by using Data-Fit software to correlate the relation between m_v and D, an empirical equation can be deduced for the decreased value of a_v with the increase of D, as the following:

$$\Delta m_v = m_v * (-A * D + B)$$

Since:

m_v is the volume change coefficient (mm^2/kg)

Δm_v is the decreased value of the volume change coefficient (mm^2/kg)

D is the cohesive soil sample depth from ground surface

A&B are constant numbers

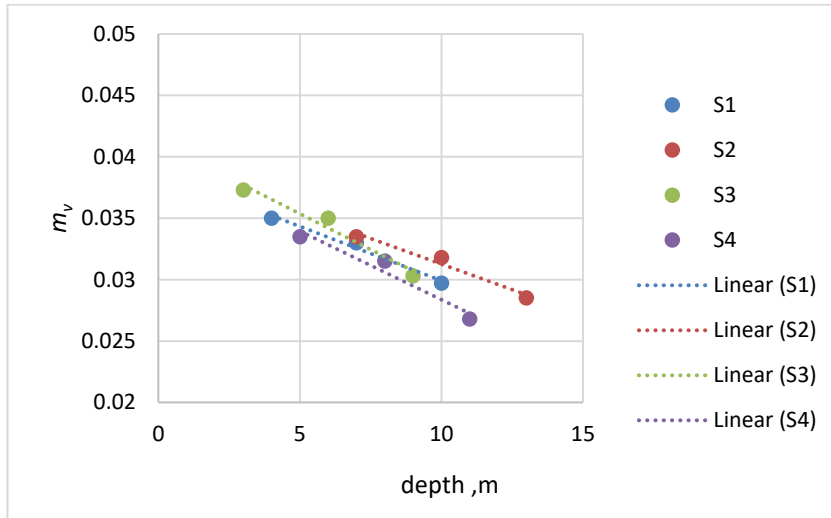


Fig. 8: Relation between m_v and D at variant sites

Accordingly, A and B values due to the fitting of the relations between m_v and D, it can be noted that:

A. At site 1, the fitting data can be as:

$$\Delta m_v = m_v * (-0.0009 * D + 0.0388)$$

B. At site 2, the fitting data can be as:

$$\Delta m_v = m_v * (-0.0008 D + 0.0396)$$

C. At site 3, the fitting data can be as:

$$\Delta m_v = m_v * (-0.0012 D + 0.0412)$$

D. At site 4, the fitting data can be as;

$$\Delta m_v = m_v * (-0.0011 D + 0.0395)$$

In general, the value of constant A and B for the studied cohesive soil layers at variant sites are about (-0.0009 to 0.0011) and (0.0388 to 0.0412) respectively. So, the empirical equations can be applied as:

$$\Delta m_v = m_v * [-(0.0009 - 0.0011) D + (0.0388 - 0.0412)]$$

IV. CONCLUSION

The following conclusions are related to the analysis of consolidation parameters that obtained from experimentally laboratory applied tests of consolidation for studied soils. Accordingly, it can be drawn that

1. The average values of natural soil density (γ_{nat}) at sites for D3 are more than that for D1 and D2 by about 3.65% and 1.48% respectively.
2. The average values of natural water content (w_{nat}) at sites for D3 are less than that for D1 and D2 by about 11.14% and 5.68% respectively.
3. The average values of compressibility coefficients (a_v) for sites at D3 are less than that for D1 and D2 by about 20.29% and 13.72% respectively under each applied stress increment.
4. The relation between compressibility coefficients (a_v) and depth (D) is suggested by the following empirical formula:

$$\Delta a_v = a_v * [-(0.0019 - 0.0025) D + (0.072 - 0.0759)]$$

- The average values volume change coefficients (m_v) for sites at D3 are less than that for D1 and D2 by about 17.13% and 12.19% respectively under each applied stress increment.
- The relation between volume change coefficients (m_v) and depth (D) is suggested by the following empirical formula:

$$\Delta m_v = m_v * [-(0.0009 - 0.0011) D + (0.0388 - 0.0412)]$$

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Converting Egyptian Commercial Building to NZEB using Low-Carbon Measures

Ayman Mohameda¹, Moataz O. El-Sherif¹, Mohamed Fatouh^{1,2}

¹ Mech. Power Eng. Dep., Fac. of Eng. @ El-Mattaria, Helwan Uni., Masaken El-Helmia, Egypt.

²High Institute for Eng. & Tech. - Obour, k21 Cairo/Belbies Rd, Cairo, Egypt,

Corresponding Author Email: ayman.a.m.mohamed@m-eng.helwan.edu.eg

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Abstract – The present work aims at analyzing the energy savings of the low-carbon retrofitting measures in an existing administration building as well as their economic feasibility. At starting, energy consumption analysis of an exemplary existing commercial building in Cairo (Egypt) is carried out. Then, low-carbon measures in the considered building are applied and evaluated. The low-carbon measures include both passive and active measures to reach Net Zero Energy Building (NZEB). The existing building energy consumption of heating, cooling, ventilation, lighting, and appliances are analyzed using building simulation package, which consists of Sketchup, Open Studio and Energy Plus software under Cairo climatic conditions. The simulated results are validated with the real energy consumption of the existing building. The yearly average deviation between the simulated results and the real existing case is less than 1%. The economic feasibility of all implemented low carbon measures are conducted using the payback period. The reported results confirm that converting the existing administration buildings to NZEB is applicable under the Egyptian circumstances using low-carbon retrofitting measures implementation and renewable energy (PV panel) systems installation.

Keywords – commercial building; net zero energy building; low-carbon measures; cooling demand, building energy modeling and simulation.

I. INTRODUCTION

Decreasing energy consumption, carbon emissions and water usage are main keys toward better environmental sustainability. The increasing energy consumption and the accompanied greenhouse gas emissions are considered one of the greatest concerns recently. The total electricity demand in Egypt for the year 2018 reached 131.15 TWh with an annual average growth rate of 4.12%. The building sector alone contributes with 68% of total electricity demand in 2018 (EEHC, 2018).

In MENA region, the proposed energy saving strategies were applied on cases to develop the building regulations and energy codes (Hanna, 2010; Hanna and Physicist, 2006; Shamseldin, 2017). The National Energy Efficiency Action Plan (NEEAP) in Egypt targets to apply 3 steps to reduce electrical energy consumption: efficient lamps (LED), efficient appliances and the encouragement to use solar water heaters (Elrefaei and Khalifa, 2014). Green pyramid rating system (GPRS) was developed for Egypt as a national environmental rating system for buildings but it is not certified yet (Hanna, 2015). Using a model building simulation tool and making sensitivity analysis to reduce the electric power consumption on peak hours was carried by (Elharidi et al., 2013). The effect of implementing passive low-carbon measures on new buildings was studied theoretically in several studies. The shading, window glazing, air tightness and insulation can reduce energy consumption of an average of 33% (El-Darwish and Gomaa, 2017). Regarding active low-carbon measures, the high investment in the solar panels could be achieve a negative final energy balance and the feasibility of applying nearly zero energy building (nZEB) as a proposed solution for the energy problem in New Borg El Arab City, Egypt (Reda et al., 2015). Solar panels could be used also in a hybrid renewable energy system to achieve a net zero energy village in Alexandria (Diab et al., 2015).

Accordingly, the national trend and the target of this phase is going to reduce the energy consumption in the buildings sector by applying the NZEB strategies to the new buildings and refurbishment the already

existing buildings to reduce the demand and apply on-site generation to reach NZEB target. By 2030, the sustainable development strategy (Egypt vision 2030 – Second Pillar) aims at reducing the energy consumption and greenhouse gas emissions by 14% and 10%, respectively.

The objective of the present research is to evaluate of applying energy efficiency measures on existing administration building under Cairo climatic conditions to reduce both energy consumption and greenhouse gas emission.

II. RESEARCH METHODOLOGY

The research methodology includes selected building profiles and simulation tools as well as models of existing and retrofitted buildings. All architectural and performance data are obtained from the designer and the operator. Thereafter, the model is created on a certified building simulation tool. The simulation results are verified with the actual energy performance. The proposed passive and active low carbon measures are implemented on a simulation model to examine the techno-economic performance of the building.

Selected Building Overview

The selected building includes a basement, ground level and four typical floor plans, as shown in Figure 1. The building locates in Maadi smart village (29°58'19" N, 31°17'00" E), Cairo, Egypt. The building covers an area of 4000 m² and the parking lot is 2,107 m². The total building area and height of the building are 25,000 m² and 28.5 m, respectively. The area with air conditioning is 19,342 m² (77.4%), while the area without air conditioning is 5,658 m² (22.6%).

Building Simulation Tool and model

Open Studio V.2.3, Sketchup V.17.0 and Energy Plus V.8.8 is used as an integrated package to simulate selected buildings. This package is an open-source platform that has been used in several published investigations (Guglielmetti et al., 2011; Raslan and Mavrogianni, 2013).

2.2.1 Existing building (reference case)

The existing building envelope thermal properties are obtained from the design package (ECG, 2012). Its external walls consist of sandstone (hashma stone), air gap, insulation board, vapor barrier, and hollow block masonry. The overall heat transfer coefficient (U-Value) of the wall is 0.28 W/m²K. The solar heat gain coefficient (SHGC), the heat transfer coefficient (U-value, and the window-to-wall ratio of the building window are 0.3, 1.44 W/m²K and 25%, respectively. The design year and weather file for Cairo, Egypt is used (Energy Plus, 2021).

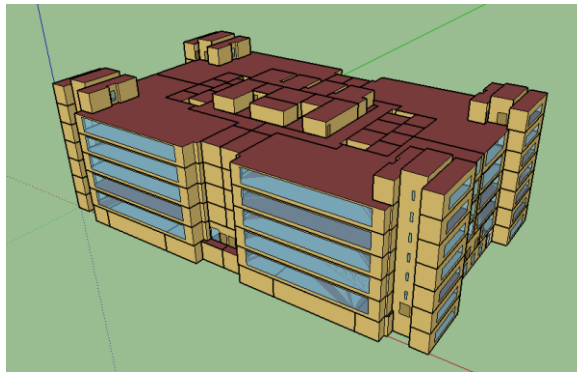


Figure 1 Case Study Building Model

The existing building consists of 251 spaces that can be classified into certain types based on activity usage reported in the literature (Brackney et al., 2018). Air-conditioned space types are call centers, training rooms, rest areas, lockers, corridors, and IT rooms. Other space types are only ventilated, such as storage rooms, electrical rooms, and toilets. The building is operating for 24 hours on three consecutive work shifts (each, 8 h). With 3,093 building occupants per shift, there are no temporary staff, visitors, or residents in the building. Each space type has an occupancy schedule based on building operations work shifts starting at 8am, 4pm and 12pm each day, as shown in Figure 2. As shown, these schedules are being changed to half on weekends and official holidays. According to the design, the peak occupancy load is 8.22 m²/person. Table 1 gives the peak lighting power densities according to the literature (Schwedler et al., 2010) and the design package (ECG, 2012). The electrical equipment load is obtained from the distribution of equipment in all spaces in the building considering the capacity and quantity of each equipment. The ventilation system brings fresh air into the building, with an average of 0.3 L/s m² according to the design package.

The existing building uses a variable refrigerant flow (VRF) air-conditioning system with cooling capacity of 1050 TR. The VRF air-conditioning system has various advantages such as sustainable, cost effective, lower maintenance costs and quiet operation. As important, VRF technology offers the ability to capture a significant number of points toward LEED certification. The VRF air conditioning system in the existing building uses R410a as the working fluid and has a rated cooling COP of 3.3 and a rated heating COP of 3.5.

2.2.2 Modified building using low-carbon measures

Existing buildings are retrofitted with low-carbon measures, which can be divided into passive and active measures. Passive measures include increasing external wall insulation, window shading, and choosing window types with low solar transmittance. To replace high-power lamps, energy-saving lighting lamps (fluorescent lamps, LED lamps, etc.) are used as active measures. The effect of implementing these measures on the existing building are investigated and compared with a reference model (existing building). Also, the impact of applying all measures to the base case model is studied and compared with the reference case.

The existing type of glazing in the existing building is double glazing with air space. It has a U value of 1.44 kW/m²K, good reflectivity, moderate light transmittance and moderate solar heat gain. Other window glazing types are proposed to investigate their effect on annual thermal demand. Table 2 lists thermal and optical characteristic of the proposed glazing types.

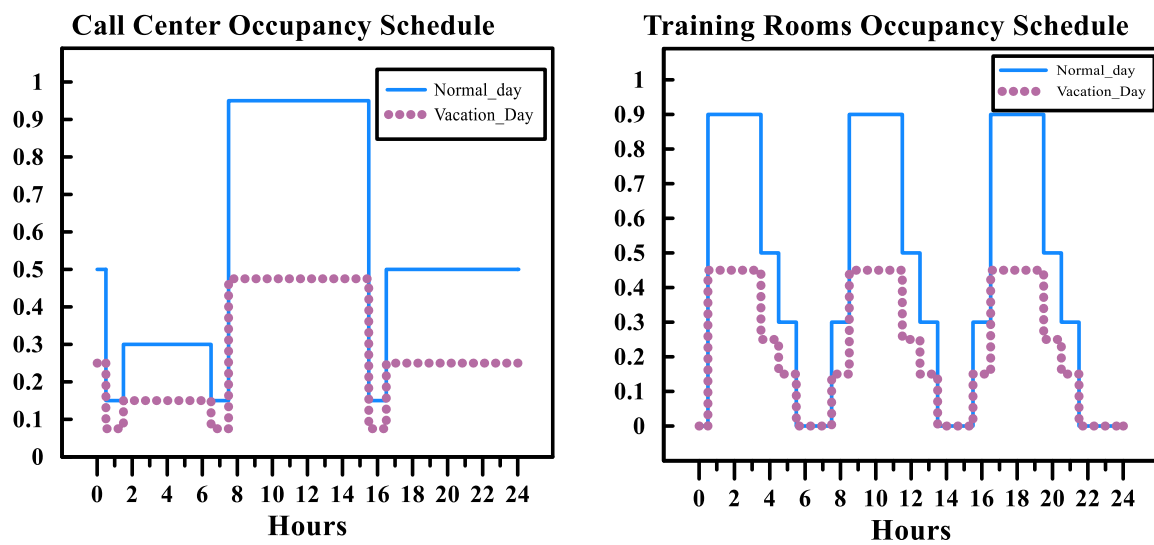


Figure 2: Occupancy Schedules

Table 1 Base Model Lighting Intensity (ECG, 2012)

Space type	Base Model lighting intensity (W/m ²)	Space type	Base Model lighting intensity (W/m ²)
Break Area	6.69	Stairs	6
Call Center Room	12	Storages	9
Corridors	5	Training Rooms	12.78
IT & Server Rooms	13.83		

Table 2 Thermal and optical characteristic of glazing types (Hanna, 2010; HBRC, 2006)

Types	Name	U	SC	SHGC	τ_{vis}
00	Base Model	1.44	0.42	0.3	0.45
01	Single Glazing (Blue)	6.17	0.71	0.61	0.57
02	Single Glazing (Grey)	6.17	0.69	0.59	0.43
03	Single Reflective (Class A) 1 Clear High Emissivity	5.41	0.36	0.31	0.2
04	Single Reflective (A) Tint Medium Emissivity	5.11	0.29	0.25	0.09
05	Double Glazing Tint Low Emissivity	1.78	0.37	0.28	0.44
06	Double Glazing, Reflective (A) Clear Medium Emissivity (IG)	2.35	0.2	0.17	0.13
07	Double Glazing, Reflective (A)Tint, Medium Emissivity (IG)	2.35	0.18	0.15	0.08

Where U is the U -value [$\text{kW/m}^2 \text{K}$], SC is the shading coefficient [-], $SHGC$ is the solar heat gain coefficient [-] and τ_{vis} is the visible transmittance [-].

The most popular passive method of controlling sunlight and unwanted solar heat is external shading. The unwanted solar rays can cause occupant discomfort and increase cooling loads. The projection factor is a way of characterizing the effect of horizontal shading on facade windows. The projection factor (PF) is defined as the ratio between the length of the horizontal shadow and the height of the window. In the base model, the facade windows are exposed to sunlight. The projection factors of 0.4, 0.6, 0.8, and 1.0 are used in each direction separately as well as in all directions together.

Based on the existing lighting system data, the lighting lux of the space type is estimated according to HBRC (2006). The present work includes 4 types of lighting fixtures. It may be noted that fluorescent lamps are considered as the base model lighting fixtures. Alternative lighting fixtures include incandescent, compact fluorescent (CFL), and light-emitting diode (LED) lamps. These alternative lighting fixtures are investigated with the design lighting lux level remaining constant as in the base case.

Lighting efficacy of incandescent and CFL is 17.4 and 78.9 lm/W, respectively, based on the reported data in literature (HBRC, 2006) while that of fluorescent lamps and the alternative LED lamp is 60 lm/W and 106.67 lm/W, respectively, according to manufacturer catalogue (Philips, 2019). The lighting intensity of each alternative lighting fixture is calculated by maintaining the same light intensity (lux/ m^2) in all spaces.

2.2.3 Adding photovoltaic panels to the Modified building

The photovoltaic panels are proposed to be fixed on the existing building, with a total available surface area of 5,615 m^2 for the installation of photovoltaic panels. The total surface area includes 3,508 m^2 of roof plan area (about 90% of the roof area, which is reserved for maintenance purposes) and 2,107 m^2 of off-building parking. The efficiency of the photovoltaic panel and the inverter is 18% and 95%, respectively, according to the manufacturer data (ABB, 2019; Jinko, 2019).

2.2.4 Economic analysis of low-carbon measures

The economic analysis of low-carbon measures is carried out using the payback period method. The payback period is the ratio of the increase in the total investment cost of the retrofit (i.e., from a less efficient design to a more efficient design) to the reduction in annual operating costs. This type of calculation is known as a “simple” payback period because it does not consider changes in operating expenses over time. As noted in literature (Shouman et al., 2016), projects with short payback periods are less risky. The payback period is calculated in two steps using the above definition. The first step is to estimate the payback period for each energy efficiency measure, and the second step is to calculate the payback period for PV panel installation.

The input data to the program are the weather file and design days, the building operation schedules, the building construction and used materials, the building loads (people, lighting, equipment, etc.), the space types, thermal zones, HVAC systems, OpenStudio measures, PV data and economic parameters. Output data from the program includes Open Studio reports that include net site energy, annual and monthly energy consumption, total facility building electricity consumption, photovoltaic panel output, and payback period.

III. RESULTS AND DISCUSSION

In this section, the results of the base case model are reported, discussed, and validated against the actual energy consumption obtained from the building database. The impact of proposed low-carbon measures and photovoltaic panels is predicted, followed by their economic performance.

3.1. Base Case Results

Hourly space cooling and heating demand is shown in Figure 3. Based on a total floor area of 25,000 m^2 , the peak space cooling and heating requirements are 1.25 MW and 30 kW, respectively. Figure 4 shows the monthly demands. The annual space cooling and heating requirements are 171.2 kWh/m^2 and 1.1 kWh/m^2 , respectively. The peak monthly space cooling demand occurred in August at 21.16 kWh/m^2 . The annual electricity consumption is 107.71 kWh/m^2 , of which the annual equipment load is 78.6 kWh/m^2 , and the annual lighting load is 29.11 kWh/m^2 .

3.2 Building Model Validation

To validate the model using the actual energy consumption of the building, the space heating and cooling demands must be converted into electricity delivery demands, resulting in the air conditioning system described earlier in Section 2.2.1. The simulated total annual electricity consumption including the air-conditioning system, equipment and lighting loads is 200 kWh/m^2 . According to the building's meter reading, the existing building consumes 198.6 kWh/m^2 of electricity per year. The monthly average deviation between the simulation model and the actual building for the whole year is 1.36%. The deviation is due to inaccurate weather files,

human error in meter reading, and unmeasured changes in actual occupancy rates, and the data obtained are monthly data. The simulated and real monthly energy demands are shown in Figure 5. It can be concluded that the simulation model is reliable and suitable for predicting the effect of implementing the proposed low carbon measures.

3.3 Results of the Modified building

External shading of windows and changing window types are considered as passive low-carbon measures, and high-efficiency lighting (fluorescent lamps, LEDs, etc.) are used as active low-carbon measures to retrofit buildings. Moreover, photovoltaic panels are installed to reduce electricity demand and achieve NZEB balance. The impact of implementing low-carbon measures on the base model is investigated and compared with a reference case.

1) Effect of the Shading

Figure 6 illustrates the cooling and heating requirements for the use of external shading on windows. Since the building's orientation and sun position face the building's south façade most of the year, adding shading to the south can significantly reduce cooling requirements. Similar results were found by Elhadad et al. (2018). Vice versa, north-facing shades have the lowest cooling demand reductions. When the shade is installed in all direction, the cooling load is reduced by 5.3% and the heating load is increased by 0.77%.

2) Effect of the Glazing Type

Figure 7 shows the annual cooling and heating demands due to the installation of the proposed glazing types. The lowest cooling load is achieved with glazing type 07. The highest cooling loads occur when using type 01 glass. Type 04 glass, on the other hand, produces the highest heating load, and solar radiation easily passes through the glass due to its high solar coefficient (SC) and high visible light transmittance (τ_{vis}). Due to the very low visible light transmission (τ_{vis}), the heating load is the lowest when using type 01 glass.

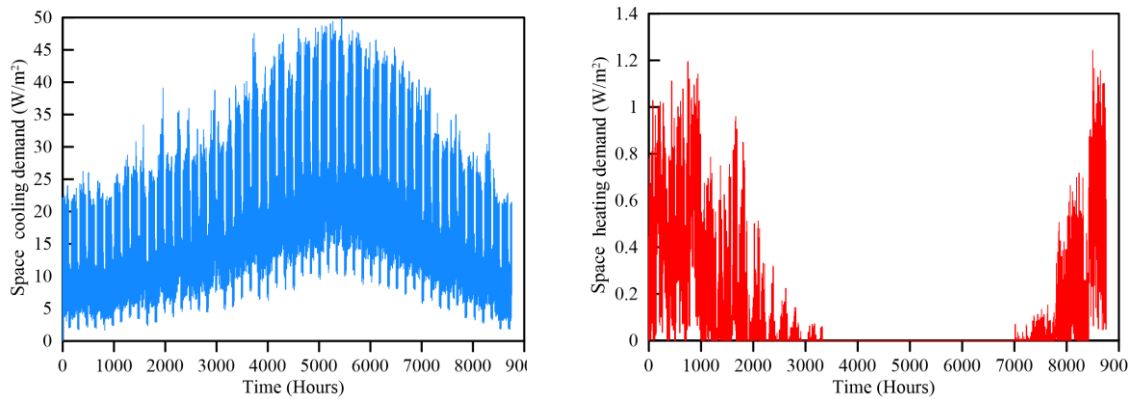


Figure 3 Base case annual space cooling (left) and heating (right) demand

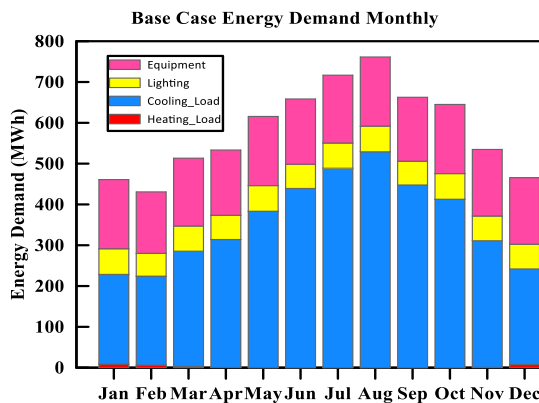


Figure 4 Base case monthly demand

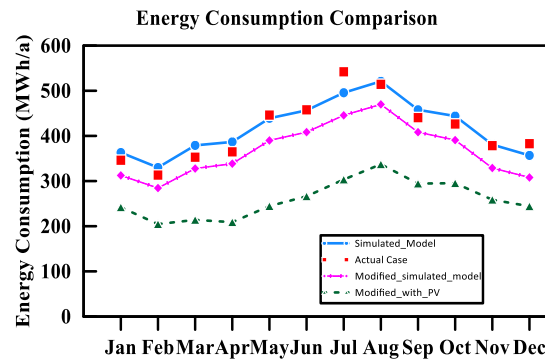


Figure 5 Monthly Energy Demand of basic model and modifications

3) Effect of the Lighting fixtures

Figure 8 presents the space cooling, heating, and electric demand using different lighting types. LED lighting has the lowest cooling demand while the incandescent lighting has the highest cooling demand. On the other hand, the lowest heating requirement is achieved with incandescent lamps, as their low efficacy of 17.4

lm/W results in high internal heat gain. The highest heating demand is achieved with LED lighting, as its high efficacy of 106.67 lm/W results in the lowest internal heat gain. Compared to the base case, using LED lights reduces cooling requirements by 7.9%, increases heat load by 5.5%, and saves 43.8% in primary electrical lighting requirements. Regarding the total electricity demand delivered, implementing LED lighting achieves a total energy savings of 9.4% compared to the base case.

4) Effect of Photovoltaic panels

The suggested locations for these photovoltaic panels are the roof and the parking lot outside the building. The available areas of the roof and the parking lot are 3,508 m² and 2,107 m², respectively. The simulation results show that the photovoltaic panels generate 1.3 GWh/a in a total area of 5,615 m² (that is, each square meter of photovoltaic panels produces 230.8 kWh/m² per year). The efficiency of photovoltaic panels commonly used in the local market is assumed to be 18%. The active area accounts for 90% of the total photovoltaic area. The inverter efficiency is assumed to be 95%.

Figure 5 shows the monthly electricity demand for the base (actual) case and the modified model without and with PV panels. The modified model includes integrated glass type 07 glazing (double glazing, reflective (A) tint, medium emissivity (IG)), LED lighting type, and south shading with a projection factor of 1.0. The simulated annual electricity demand of the modified model is 176.4 kWh/m². After applying photovoltaic panels, the annual electricity demand is reduced to 124.5 kWh/m² (about 29.45% reduction). However, the annual energy consumption of the building implementing all the proposed modifications is reduced by 37.86% compared to the actual building electricity bill. To be clear, to achieve NZEB, the required photovoltaic panel area must be equal to 13,489 m². It is about 3.37 times the footprint area of the building. This is quite unachievable due to whether the land availability in the existing building location is limited, or the land cost is very expensive.

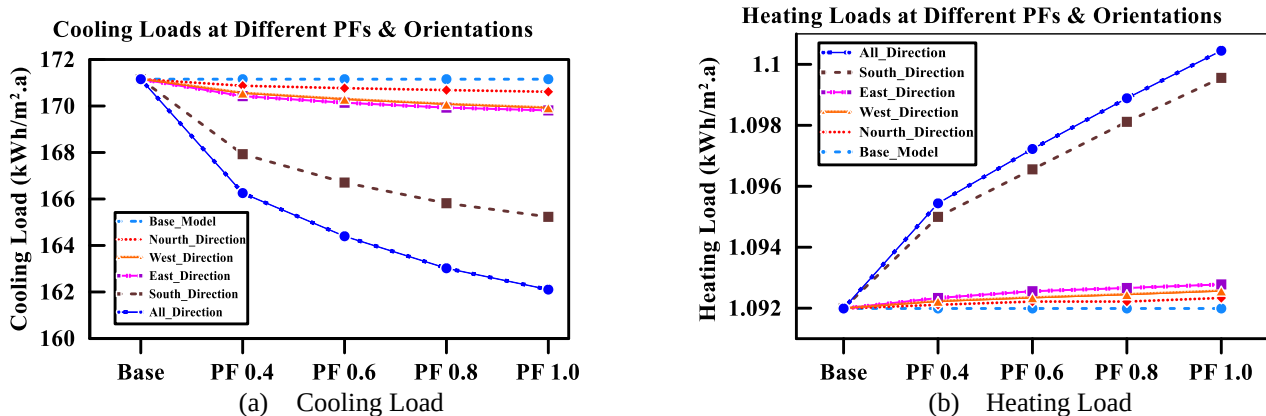


Figure 6 Cooling and heating Loads at Different PFs and Orientations

5) Payback period results

The initial cost of energy efficiency measures (glazing, shading, LED lighting) and photovoltaic panels is estimated based on local market research. Operation and maintenance costs are estimated based on the life and maintenance schedule of each measure. Glass and shades have a long lifespan of about 20 years. This means there is no cost to change it throughout its lifetime. Based on the manufacturer's recommendations, change of both LED lighting and photovoltaic panels is taken into consideration throughout the life of the project.

The simulation results confirm that the total glazing (Type 07) area that had to be supplied and installed is 2487 m² ($\approx$ 2500 m²). According to the local market prices, it is found that the cost to supply and install (cladding and frame) of this glazing type (Type 07) with shading has an average price of 1,000 LE per m². Hence, the glazing replacement and shading installation cost is 2,500,000 LE.

The main types of lighting fixtures are 18W compact, 14W 600mm tube fluorescent and 28W 1200mm tube fluorescent. These types will be replaced by 9W LED bulbs, 8W LED 600 mm tubes and 14.5W LED 1200 mm tubes. Each space in the building has its lighting operating hours. This helps to calculate how many times each lighting fixture allocated in each space will be replaced. The 8W 600mm LED tube and 14.5W 1200mm LED tube only need to be replaced once in the 20-year project period. The 9W LED bulb will be replaced three times. Hence, the total cost of the lighting equipment is 1,794,800 LE.

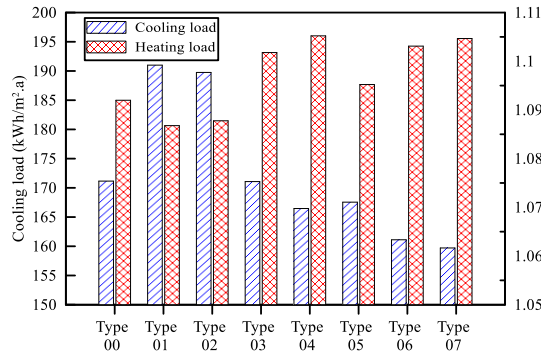


Figure 7 Annual Cooling and Heating Loads at different Glazing Types

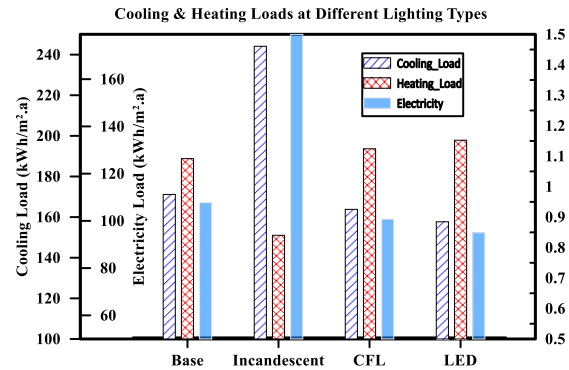


Figure 8 Cooling & Heating Loads at different Lighting Intensity

Existing lighting fixtures have lifetimes of 6,000, 20,000 and 20,000 hours. The replacement cost of the base model lighting fixture is 1,253,924 LE. Therefore, the net cost of implementing efficient lighting measures is 509,924 LE.

According to the simulation results, the solar panels produce 1,296,191 kWh/a. According to local market research, the average cost of 10 m² of solar panels is 17,000 LE. Maintenance costs are assumed to be 1.5% of capital cost per year. Therefore, the total initial and maintenance cost of the PV is 10,355,516 LE.

The simulation results revealed that the total initial and replacement cost is 13,365,440 LE for all low carbon measures and photovoltaic solar panels, and the operating cost saving is 3,035,046.4 LE. Consequently, the payback period for the retrofit of the building is 4.4 years.

IV. CONCLUSION

The present work aims at analyzing the energy savings of the low-carbon retrofitting measures in an existing administration building as well as their economic feasibility. The simulated results of an existing administration building showed that the annual delivered electricity is 279.955 kWh/m². The effects of implementing low-carbon measures can be summarized as follows:

- Replacing the base case glass type with a double-glazed, reflective (A) Tint, Medium Emissivity (IG) (type 07) provides a 4% reduction in energy compared to the base case.
- The percentage of energy savings due to the addition of shading to the south-facing façade windows is 2.11%.
- After applying 3 lighting fixtures; LED, CFL and incandescent lamps, LED lighting provides a saving of 43.8% in electricity demand for lighting and 9.4% in total electricity demand.
- The modified model integrating all low-carbon measures (except photovoltaic systems) achieved 11.9% savings in annual electricity demand relative to the base model.
- By installing a photovoltaic panel system on the modified model, the delivered energy demand is reduced by 29.45% compared to the base case.
- Achieving a net-zero energy building requires 3.37 times the building footprint.
- The payback period of the modified case without the PV system is 3.15 years. Adding a photovoltaic system to the modified model extends the payback period to 4.4 years.

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