A STATE OF THE ART REVIEW ON REINFORCED CONCRETE VOIDED SLABS

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ABSTRACT

Voided slabs remove the excess concrete from the unnecessary part of the structure slab to reduce weight. It was invented in Denmark about twenty seven years ago. It is now gaining popularity in the world. This paper reviewed several studies done on voided slab system. Every specialized parameter of voided slab system on which exploratory review has been done by author is tabulated in this paper systematically. The realization of the proposed goals includes documentation action and theoretical investigation of all work done by a few creators on voided section idea. The resultant conclusion will be used in defining the falling mechanism that can be helpful in the producing of a sufficient numerical model.

Keywords: reinforced concrete, review, voided slabs, experimental, theoretical.

1. INTRODUCTION

Voided slab systems are an alternative to conventional concrete slab construction. Voided slab systems are lighter than solid concrete slabs while maintaining the ability to have large spans. Because of less concrete quantity is used in voided slab construction, slabs are lighter than traditional. This section research how voided slabs are constructed and how lighter weight and larger spans influence on building design. When designing a reinforced concrete structure, a primary design limitation is the span of the slab between columns. Designing large spans between columns often requires the use of support beams and/or very thick slabs, thereby increasing the weight of the structure by requiring the use of large amounts of concrete. Heavier structures are not preferred because of larger dead load for building increases the magnitude of inertia forces that the structure must resist as large dead load contributes to higher seismic weight. Incorporating support beams can also contribute to larger floor-to-floor heights which consequently increases costs for finish materials and cladding.

2. VOIDED SLAB CONSTRUCTION

Voided slab construction removes concrete in locations where the slab that is less critical to resist the applied loads. Keeping overall depth of the section with removing concrete from the slab interior allows for comparable utility in most applications as the section modulus and stiffness are roughly equivalent to a solid slab, but the self-weight of the section is greatly reduced. This reduction has many benefits. Voided slab systems were first invented in Europe in the 19th century. Since that time, many European companies have patented their own systems. As a result, most uses of voided slabs have occurred in Europe. Many types of voided slab brands, this section briefly reviews all major types of voided slab and the slight differences that exist between them.

2.1 Bubble deck

In the middle of 19th century, a new system was invented in Denmark by Jorgan-Breuning more than 30% reduction of dead weight and allowing large spans between supports which is called bubble deck system. Bubble deck is based on new patented technique which involves the direct way of linking air and steel to creating a natural cell structure acting like a solid slab. For the first time, bubble deck with the same capability as a solid slab, but with considerably less weight due to elimination of superfluous concrete. In this technology, it locks ellipsoid between the top and bottom reinforcement meshes, thereby inventing a natural cell structure acting like a solid slab. To replace the superfluous concrete, HDPE hollow spheres are used at the center of slab Plate-1 shows the Bubble Deck voided slab system (Dwivedi *et al.*2016)[1].

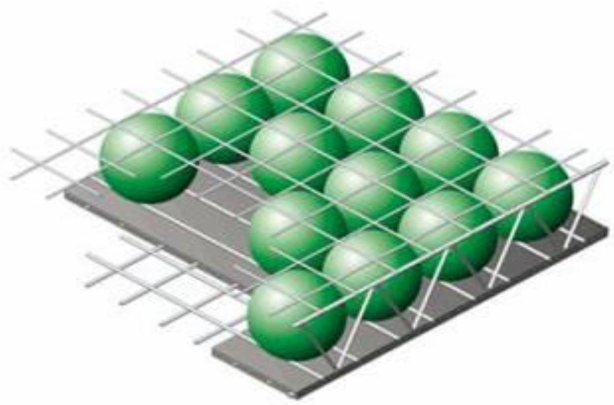

Plate-1. Bubble deck slab(Dwivedi*et al.*,2016) [1].

2.2 Cobaix

The same hollow slab principle of creating voids within the concrete slab to lighten the building structure was developed in 1997 South Africa, which was called cobiax system. Although the cross section of the cobiax is more complex as compared to the solid slab and flexural design possess no significant problems. However, when considering design for shear, the spherical void formers used in the cobiax system in concrete wave width that not only change depth of the section, but also in horizontal direction. No design code of practice has specific design recommendation for cobiax system. Extensive research on cobiax shear resistance was carried in Germany. In this

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system, decks form the bottom of the slab, and the bottom layer of reinforcing steel must also be placed. The voids are covered by steel wire meshes which can be altered to fit the particular application. The top layer of steel reinforcement can be placed after the bundles are in place. Concrete is then poured in two lifts. The first concrete pour covers the bottom reinforcement and a portion of the voids and holds the voids in place as the concrete becomes stiff. The second lift is poured after the first lift is stiff but still fresh, finishing the slab. This method requires more formwork and on-site labour, but requires less transportation of materials.Plate-2 shows the Cobiax voided slab system (Dwivedi *et al.*, 2016) [1].

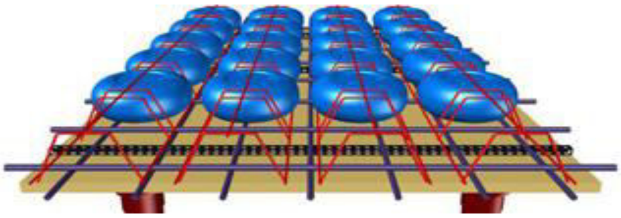

Plate-2. Cobaix voided slab system (Dwivedi, *et al.*2016) [1].

2.3 U-Boot Beton slab

A new invention system of hollow formers in order to decrease the transportation cost and CO_2 production was patented in 2001 by an Italian engineer, Roberto llGrande. U-Boot Beton, or U- boot, is a voided slab system from the Italian company Daliform. Spherical void does not used in U-boot like previous systems, but uses truncated-pyramid shaped void formers instead. These void formers create many grid shaped beams making up the slab (U-boot Beton, 2011). There is similarity between U-bootand Cobiax system in terms of construction because it is meant to be cast entirely on-site using formwork. After forms are erected, the steel and void formers are placed before the concrete is poured in two lifts. The U-boot system has single benefit over all previous benefits of voided slab systems that provide systems used spherical void formers. The shape of the U-boot void formers allows them to be stacked efficiently during transportation to the site, saving space and potentially leading to reduced shipping costs compared to spherical former systems.Plate-3 shows U-boot voided slab system. (Dwivedi, *et al.*2016) [1]

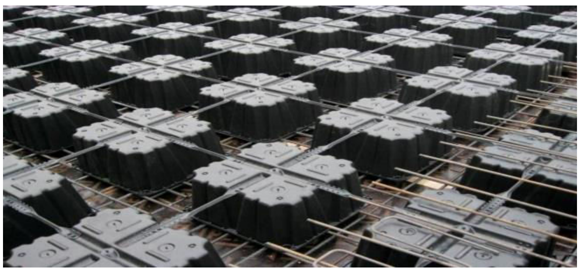

Plate-3. U-boot voided slab system(Dwivedi *et al.*, 2016) [1].

2.4 Ribbed (Waffle) slab system

Equally spaced ribs are usually supported directly by columns called ribbed floors. Ribbed slab is one-way spanning systems while waffle slab is two-way ribbed system. This type of slab is not very popular because of the formwork costs and the low fire rating. A 120-mm-thick slab with a minimum rib thickness of 125 mm for continuous ribs is needed to achieve a 2-hour fire rating. A rib thickness of greater than 125 mm is usually needed to accommodate tensile and shear reinforcement. Ribbed slabs are adequate for medium to heavy loads, and can span over reasonable distances, and are very stiff and particularly suitable. Slab depths typically vary from 75 to 125 mm and rib widths from 125 to 200 mm. Rib spacing of 600 to 1500 mm can be used. The overall depth of the floor typically varies from 300 to 600 mm with overall spans of up to 15 m if reinforced, longer if post-tensioned. The use of ribs to the soffit of the slab reduces the quantity of concrete and reinforcement and also the weight of the floor. The saving of materials will be offset by the complication in formwork and placing of reinforcement. However, formwork complication is minimized by using of standard, modular, reusable formwork, usually made from polypropylene or fiberglass and with tapered sides to allow stripping. For ribs at 1200-mm centers (to suit standard forms) the economical reinforced concrete floor span 'L' is approximately D x 15 for a single span and D x 22 for a multi-span, where D is the overall floor depth. The one-way ribs are typically designed as T-beams, often spanning in the long direction. A solid drop panel is required at the columns and load bearing walls for shear and moment resistance.Plate-4 shows ribbed slab.(Kalaiyarasi, 2016) [2]

Plate-4. Waffle slab(Kalaiyarasi, 2016) [2].

The Advantages of ribbed slabs are:

- Savings on weight and materials
- Long spans
- Attractive suitable appearance for architectural design
- Economical when reusable formwork pans used
- Vertical penetrations between ribs are easy.

The Disadvantages of ribbed slabs are:

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- Depth of slab between the ribs may control the fire rating
- Requires special or proprietary formwork
- Greater floor-to-floor height
- Large vertical penetrations are more difficult to handle.

2.5 Hollow core slabs

A hollow core slab, also known as a voided slab, hollow core plank or simply a concrete plank is a precast slab of pre-stressed concrete typically used in the construction of floors in multi-story apartment buildings. The slab has been especially popular in countries where the emphasis of home construction has been on precast concrete, including Northern Europe and former socialist countries of Eastern Europe. Precast concrete popularity is linked with low-seismic zones and more economical constructions because of fast building assembly, lower self-weight (less material), Precast, pre-stressed hollow core slabs (PHC) slabs are among the most common loadbearing concrete elements in the world. They are commonly used in floors and roofs of office, industrial, residential, and commercial buildings. Typical slab cross-sections in Finland are shown in Figure-1. The manufacturing technique is simple. Pre-stressing strands are first tensioned above a long bed, where after a casting machine casts and compact the concrete around and above the strands. The ends of the strands are releasedafter hardening of the concrete and the long slab is saw-cut into units of desired length. Due to the special manufacturing technique no transverse reinforcement is possible. When subjected to a transverse line load the heavily pre-stressed slab units typically fail. The failure is abrupt and noisy, like a small explosion. Before failure, the failure zone is completely un-cracked, and when the first crack appears, the failure takes places immediately. This failure mode is called web shear failure and it is the subject of the following story [3].

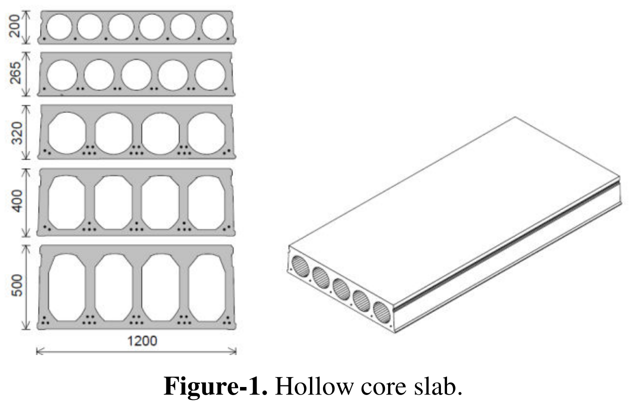

Figure-1. Hollow core slab.

3. EXPERIMENTAL STUDIES ON VOIDED SLABS

Johnson and Ghadiali (1972) [4] investigated the distribution of loads on precast hollow core slabs in areas where openings exist. The panel is made up of four precast units, connected by grouted shear keys, with a (40×40cm) opening centered in the panel. The results verified that full load distribution is attained up to the ultimate flexural capacity of the system and this study is based on a field test of hollow-core pre-stressed concrete slabs. The field test was conducted in a pre-stressed concrete plant using materials and equipment that are normally available at this type of facility. This test procedure cannot be compared with the rigidly controlled techniques of laboratory testing. It is felt, however, that field testing, when coupled with recognized methods of analysis, can provide useful information to the designer. It was showed that the system provides adequate safety factors as shown by the following observations:

a) The ultimate load requirement was met.

b) Considerable additional load was supported after first cracking.

c) Large deflection occurred as a warning that the ultimate capacity was being approached.

Oduyemi and Clark (1987)[5] carried out study on thirty specimen, fifteen 1:3.33 scale model with longitudinal circular voided slabs and fifteen 1:3.33 scale model with transverse voids. Predicting method for tension stiffening in both longitudinal and transverse voided slab strips was presented and a simplified procedure suitable for the design and predicting tension-stiffening effect of both solid and circular voided slabs is proposed. For each strip (in case of longitudinal voided) the relationships between the experimental and theoretical moment-curvature are given. The theoretical curves were obtained from the properties of the full cracked section tension stiffening force ignored. It was concluded that the maximum tension stiffening force in a longitudinal section of a voided slab is a function of the tensile strength of the concrete. And the rate of decay of tension stiffening force with strain is independent of bar spacing for a practical voided slab. This proposed method of predicting tension stiffening effect in longitudinal section of voided slabs gave good agreement with tests on fifteen slab strips. In case of transverse voided, it was concluded that the method of predicting tension-stiffening force also had been proposed which give good agreement with the tested slabs strips.

El-Behairy, *et al*. (1989) [6] conducted experimental study to explain the general deformational behavior of voided reinforced concrete slabs under different loading conditions. Six voided reinforced concrete slabs with dimensions (1.04x1.8m) having void diameters of (63, 50 and 40mm),(10 voids for each one) as shown in Figure-2.Different reinforcement percentage was tested by loading with different concentrated loads using

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simply supported on span of 1.6m. The varying parameters studied were the voids diameter and the percentage of reinforcement. These slabs were divided into two groups. The first case is loaded by single concentrated load acting at center of slab and the second group loaded with eccentricity 0.3m from the center of slab. The experimental results were higher than those obtained theoretically by about10-20 %.as shown Figure-2. It was noticed that the deflection decreases as the voids diameters decrease, and also as the reinforcement increases. The conclusions shows that the orthotropic plate theory can be used for the analysis of circular voided slab, decreasing the void-depth ratio improves the load distribution across the voided slabs.

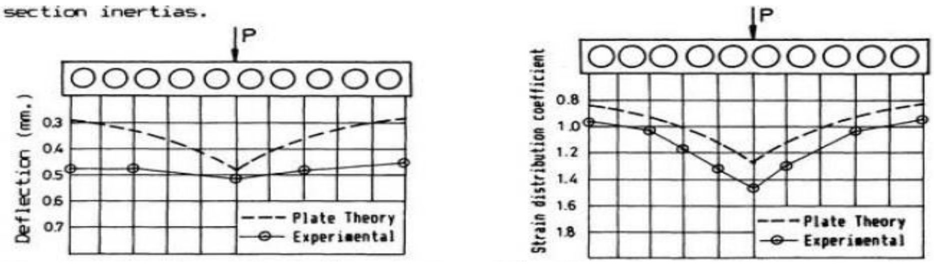

Figure-2. Deflection distribution along the mid-section.

Ueda and Stitmannaithum (1991) [7] presented the result of an experimental investigation on the shear strength of composite precast pre- stressed hollow concrete slabs with concrete topping. Neither shear reinforcement nor shear keys were provided in the specimens. The experimental variables were pre-stressing force, tension reinforcement ratio, depth to span length ratio, and thickness of concrete topping. From the test results, the prediction methods were proposed for both cracking and ultimate strength in shear. It was found that the pre-stressing force is more effective in increasing both the flexural and shears cracking strength than in ordinary monolithic pre-stressed concrete. While it is much less effective on the ultimate shear strength, and the effect of the hollow core exists only in the cracking strength but not in the ultimate strengths. The shear cracking strength can be predicted by the conventional methods for monolithic pre-stressed concrete members, assuming entire composite action. Although actual hollow shape should be considered for prediction of the web-shear cracking strength, the effective web width at the hollow level should be used for the flexure-shear cracking. Although no shear reinforcement was provided, the ultimate shear strength was significantly greater than the shear cracking strength, except for thick concrete topping

Abdul-Wahab and Khalil (2000) [8] tested eight large-scale (1/4-in. scale) waffle slab models with varied rib spacing and rib depth. The test program was designed to investigate the effect of the spacing and the depth of ribs on the flexural rigidity and strength of waffle slabs. In addition, two solid slabs were also included: one having the same overall depth as the slabs in the first group to study the flexural and torsional effects, and the second to investigate the equivalent thickness hypothesis. The test specimens were large-scale square reinforced concrete models, with a scale ratio of 1:4 which representing real spans of 6.0m. The slabs were isotropically shaped and isotropically reinforced in two perpendicular directions, so that the slab moments of resistance were identical in those two directions. As shown in Figure-3, the overall dimensions of the test specimens were 1540 x1540 mm. Details of their properties are given in Table-1.

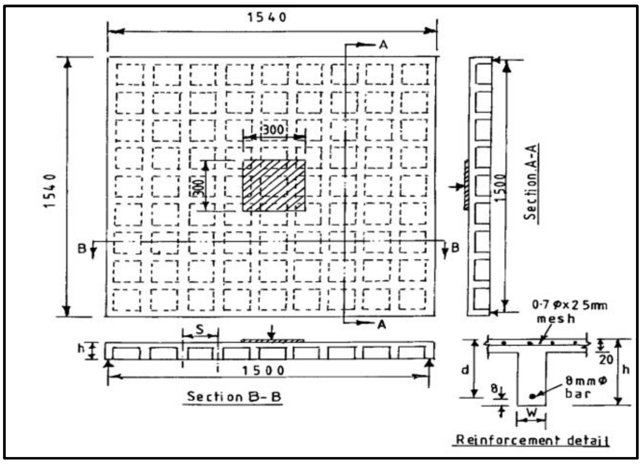

Figure-3. Geometry of waffle slabs, loading arrangement, and reinforcement details [8].

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Table-1. Properties of tested specimens [8].

Slab number (1)	Bays (2)	Spacing between ribss (mm) (3)	Thickness offlange h_f(mm) (4)	Width ofribs w (mm) (5)	Overall depth h (mm) (6)	h/h_f (7)	Steel area (mm^2 /m)(8)
S1	11x11	136	20	52	95	4.75	370
S2	9x9	167	20	52	95	4.75	301
S3	7x7	214	20	52	95	4.75	235
S4	5x5	300	20	52	95	4.75	168
S5	9x9	167	20	57	125	6.25	301
S6	9x9	167	20	47	65	3.25	301
S7	-	-	20	-	75	1	301
S8	-	-	20	-	95	1	301

The slabs were simply supported along the edges, at spans of 1500 x 1500 mm, providing free rotations perpendicular to the plane of slab, and were subjected to a central ‘‘patch’’ load over an area of 300 x 300 mm. They were designed to achieve a concrete compressive strength of about 30 MPa at 28 days. Smooth steel bars with yield strength of 398 MPaused as flexural reinforcement consisted of Ø8mm/rib. The bars were placed in the ribs with clear covers of 8 mm. As temperature and crack control reinforcement, wire mesh with a diameter of 0.7 mm and mesh size of 25 mm was used. It was located at the middle of the topping of the waffle slabs (10 mm from the top).

Control specimens of 150 x 150 x 150 mm cubes, 150 x 300 mm cylinders and 100 x 100 x 400 mm prisms were also cast with each slab specimen to determine the compressive and tensile splitting strength, modulus of elasticity, and modulus of rupture. Deflections along the centerline and the diagonal were measured using 11 dial gauges. The loads were applied in a series of increments of 2.5-5 kN, and the deflections and strain measurements were recorded. The cracks location was also recorded. The loading continued to destruction to determine the mechanism of failure. Results of the test for all samples are listed in Table-2.

Table-2. Results of tested specimens [8].

Slab number(1)	Compressive strength$f\acute{c}$(2)	Crackload P_{cr} (kN)(3)	Ultimateload P_u (kN)(4)	$P_u$predicted (kN)(5)
S1	31.3	30	105	96.3
S2	32.0	20	81	79
S3	31.4	20	65	62
S4	28.9	20	48	44.8
S5	29.9	40	120	109.7
S6	29.1	20	48	44.1
S7	36.0	30	65	58.7
S8	28.5	50	100	78.7

From the results, all specimens behaved in an elastic manner until the first crack appeared. Flexural cracking was observed at about 25-30% of the ultimate load. In most of the specimens, cracks then extended toward the corners until flexural failure occurred. Specimens S1, S2, S3 and S4 have same cross section of the ribs but the difference in their numbers, the cracking loads are same with exception of S1 which has cracking load of 30 kN, unlike what is expected because of increasing the number of ribs will increase the moment of inertia and then improve the capacity of crack moment. This is evident through the Specimen S8 solid slab as a reference specimen which has cracking load of 50 kN. The ultimate failure loads are expected to be approximately same values because the concrete in a tension zone are not affected, but the different results are possible to change the number of ribs within each specimen and then the total reinforcement is changed.

Specimens S2 and S5 have same number of ribs but different overall depth 95mm, 125mm respectively, the cracking load and ultimate load for specimen S5 greater than S2.This because the moment of inertia for specimen

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S5 greater than S2. Specimens S6 and S7 has overall depth 65mm, 75mm respectively, for this reason the ultimate failure load was low.

The reduction in self-weight between specimens S5 and S8 is about 16% with increasing 20% in the ultimate failure load for specimen S5 compared with S8. This showed the importance of reducing the self-weight by increasing the depth of the ribs in waffle slab system. More reduction in cross-section of specimen S5 to be obtained by redesign of the specimen S5 to get same ultimate failure load for the specimen S8.

Pajari (2004) [9] carried out four tests on pre-stressed hollow core slab units. The slab units, two of them 200 mm thickness with 4m span and two 400 mm thickness with 6m span, were subjected to pure torsion and together have 1200mm width. In all tests, the observed failure mode was the same as the predicted one, i.e. cracking of top flange in angle of 45º with the longitudinal axis of the slab unit. Although, the failure mode was abrupt, the slab units showed considerable ductility after the failure. None of them collapsed before the test had to be interrupted due to excessive rotation. For 400 mm slabs the torsional stiffness observed in the tests was close to that predicted by elementary calculation method, but for 200 mm slabs the predicted stiffness was 30% lower than that observed. The predicted torsional resistance was 60% and 70% of the observed resistance for 200 mm and 400 mm slabs, respectively, when the lower characteristic value for the tensile strength of the concrete was used for prediction.

Kim, *et al* (2008) [10] carried out an experimental test to study the punching shear at connection area of I-SLAB (polystyrene voids forms) with column. The critical punching shear section is located at (d/2) (d represent the effective depth of slab) from the face of column. If the column rectangular the punching shears section is rectangular. ACI-318 presents different cases of the critical punching shear section with different shapes of column section that are compatible with solid slab system as shown in plate-5 (ACI 318 code 2005 and PCA Notes on 318-05 Section 11-12) [11].

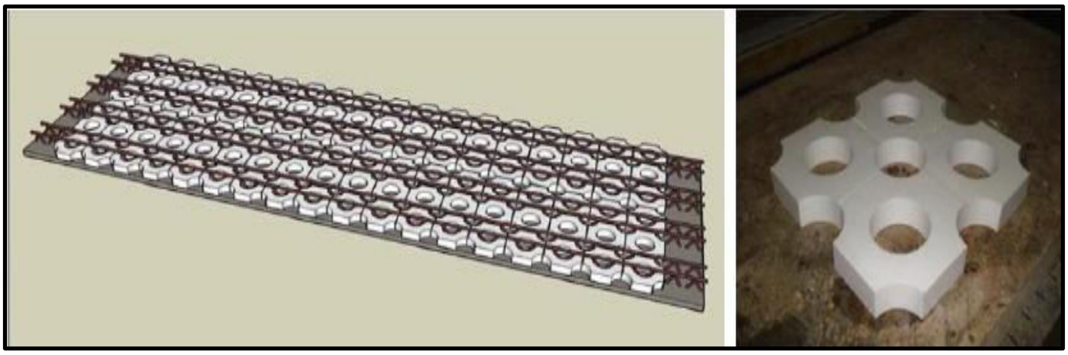

Plate-5. Polystyrene form and slab system[10].

Five interior column-slab connections in two-way slab were tested to investigate the effect of the arrangement of polystyrene form, the effect of shear reinforcements, the shapes of columns and the effect of the existence of polystyrene form.

The dimension of slab was (2200 x 2200 mm) and the slab thickness for all specimens equal to 210 mm. The polystyrene form has dimensions 400 mm length and 400 mm width with 200 mm diameter hole.All the used parameters are shown in Table-3 for the experimental test.

Table-3. List of parameters[10].

Specimens	Column section $C_1 \times C_2$(mm)	Slab thickness, h (mm)	Form type	Form arrangement situation	Shear reinforcement
P-1	600×400	210	PF1	From just column side	-
P-2	600×400	210	PF1	At a distance of 1.5H fromcolumn side	-
P-3	600×400	210	PF1	At a distance of 1.5H from column side	shear stud
P-4	400×400	210	PF1	From just column side	-
P-6	400×400	210	-	-	-

H: Slab thickness, PF1: Pocket Form Isolator (high-density polyethylene)

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The design compressive strengths of precast concrete and reinforced concrete were 30 MPa and 24 MPa, respectively. For punching shear test, the oil jack applied concentrated load equal 2000 kN at the center of column.

As result of the experimental test for all specimens, inclined crack patterns were observed by the punching shear failure. For P-2 and P-3 samples which have polystyrene form outside the distance of 1.5 H from the face of column. The failure surface moved further outside for other specimens. The P-6 specimen without polystyrene forms, the flexural cracks occurred with the inclined cracks. Also all specimens had the ultimate shear strengths around 10 mm in displacement, and they showed the brittle failures by punching shear. P-6 specimen had the greatest ultimate strength. According to the polystyrene arrangement and location, the critical sections of punching shear were changed. The summary of results is shown in Table-4.

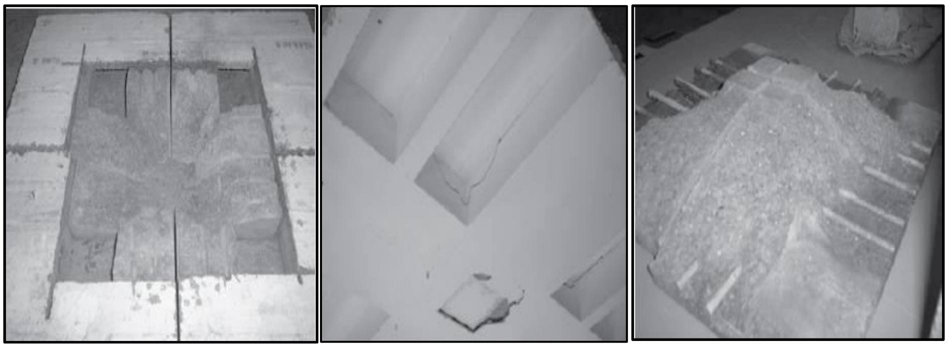
Table-4. Comparison of the test maximum strengths and the nominal strengths by ACI 318-11. [10]

Specimens	**Vtest (kN)**	**Displacement atV_{test}(mm)**	**V_{test} / V_T (V_T, Target** specimen)	**V_n (kN)**	**V_{test}/ V_n**
P-1	515.95	9.52	1.00 (P-1)	390.11	1.32
P-2	543.16	9.89	1.05 (P-1)	634.45	0.86
P-3	543.76	9.12	1.05 (P-1)	472.85	1.15
P-4	388.70	7.98	1.00 (P-4)	341.16	1.14
P-6	755.94	11.71	1.46 (P-1)	602.44	1.25

V_{test}: Maximum strengths. V_n: Nominal strengths calculated by using ACI 318-05

Lau and Obe (2011)[12]carried out experimental tests on three series (consisting of twenty-six ribbed slab samples) to determine the efficiency of the wide beams to shear failure. The first with an internal column and four equal point loads were distributed on four beams with equal distances. The second case was similar to the first case but with different loads and the third case with an edge column.

Each of the first and third cases failed suddenly by punching shear in a way very similar to the punching shear mechanism observed in the solid slabs with the failure surface inclined at about 22° to the horizon and intersecting the top surface at about 2.5 times the overall slab thickness from the face of column. However, unlike a solid slab, when the width of wide beams less than five times the overall slab depth, the failure surface was incomplete surface of revolution. This is because in this case some of the potential failure surface was lost when it entered the reduced section, as shown in Plates 6 and7 for an internal and edge column, respectively.



a. Loss of failure b- Punching failure surface c- Punched out piece

Plate-6. Internal column punching failure mechanism [12].

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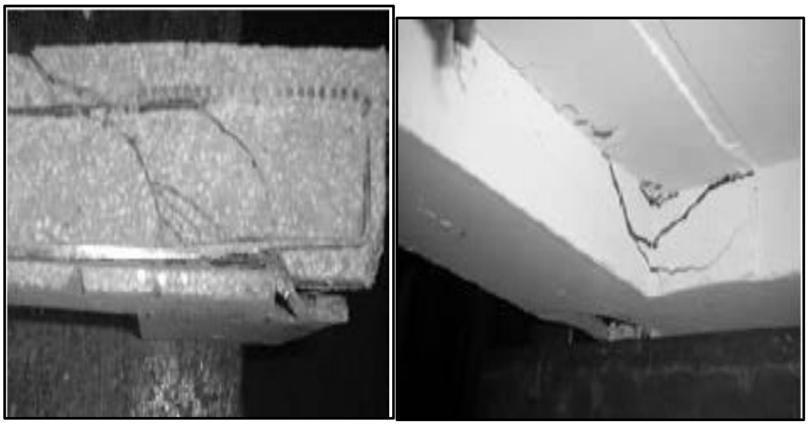
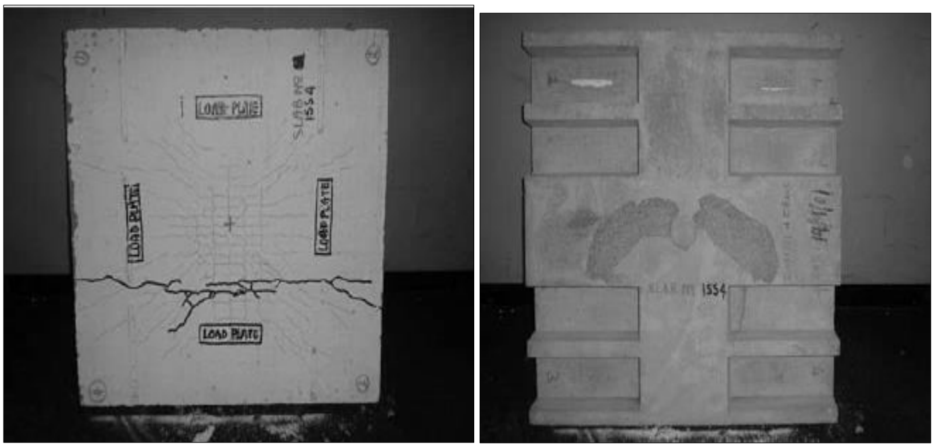


a. Inclination of internal cracks b- Loss of failure surface

Plate-7. Edge column punching failure mechanism [12].

The second case failed by wide beam shear as shown in a Plate-8.



A- Top view B- Bottom view

Plate-8. Wide beam shear failure mechanism [12].

As result of using a wide beam ribbed slab, two types of shear failure are expected, the first is punching shear and the second is wide beam ribbed slab shear. The punching shear failure of a wide beam ribbed slab is very similar to that in a solid slab but if the beam is narrow, the shear capacity is reduced relatively, because some of the potential shear failure surface is lost when it enters the ribbed section. The type of shear failure depends primarily on a width and position of the wide beam.

To simplify the calculation of shear resistance capacity and make it compatible to items of standard codes related to the shear in the solid slab it is necessary to increase the width of wide beam within the ribbed slab system.

Rahman ,*et al*. (2012) [13]tested full-scale precast prestressed hollow-core slabs (PPHC) with different shear span to depth (a/d) ratio, which were loaded to failure to ascertain the ultimate load-carrying capacity of these slabs. A total of 15 slab specimens, 5 and 2.5 m in span and having three different depths, 200, 250 and 300 mm were tested up to failure using four-point load test. It was interested to note that the failure mode of hollow-core slabs changed from pure flexure mode to flexure-shear mode for slabs with depth greater than 200 mm. The web shear cracking strength of (PPHC) slabs decreased with an increase in depth of the slab. A transition from flexure-shear to web shear failure as a function of a/d was noted in the load tests. The analysis of the experimental results showed that the existing ACI318M code equations underestimated the flexure-shear strength of these hollow-core slabs. Based on regression analysis of experimental data, a modification is proposed in the existing ACI318M code equation which can capture accurately the mode of failure and ultimate load- carrying capacity of these slabs.

Ghadiri and Marsono. (2012) [14] presented an experimental study on the effect of varying the post-stressing on the ultimate strength of post-tensioned and conventional hollow slabs to determine the actual behavior through simulation to verify, by experiment, the post-tension of hollow Industrialized Building System (IBS) slabs under various strength conditions. Two units of full scale slab with an equal span, cross section and depth were tested in laboratory up to failure with dimensions (3400mm) length, (1100mm) width and (200mm) thickness and compressive strength of concrete (40Mpa). The voids were created by PVC pipes with diameter (100mm) and (2mm) thickness. The first slab was with two post-tension bars while the other was not pre-stressed and together the slabs were subjected to two point loads over the full width of the unit at mid span as service and ultimate loading. Modeling of the slabs by using linear finite element program was done and comparison of the results with experimental was done. It was concluded that post-tension can be used to increase the bearing capacity of hollow slab by up to 15% with failure mode changes from flexural to shear as observed in laboratory experiments and ductility by 3%.

Hai, *et al* (2013)[15] studied the effects of various factors on the behaviors of bubble-deck slab such as the concrete strength, the shapes and diameters of plastic balls, the size of reinforcing mesh at top and bottom slab in order to clarify the superiority and merits of the mentioned system. The improving of the plastic ball's shape by using hollow elliptical balls for best load-bearing capacity in bubble-deck was also presented in details. The research results show the effectiveness and feasibility of the application.

The behavior of bubble-deck using traditional spherical balls and modified elliptical balls was investigated. The plastic balls have dimensions and shapes as follow: hollow spherical balls diameter of 186mm and hollow elliptical balls diameter of 240mm and height 180mm as shown in Plate-9.

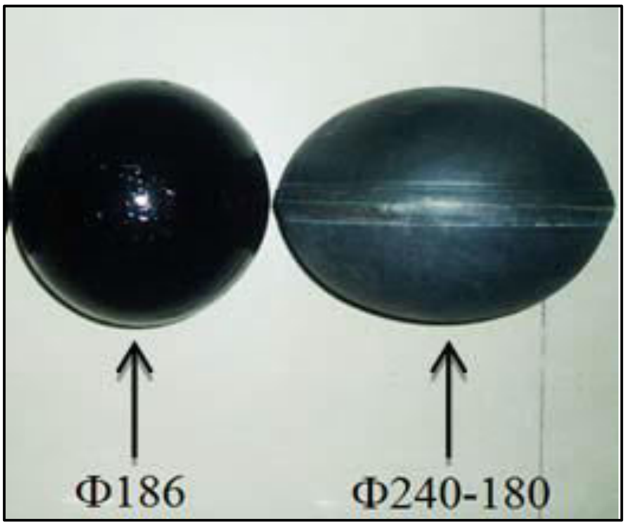


Plate-9. Shape and dimension of plastic balls [15].

Five bubble-deck samples named as A.BD.2, A.BD.3, A.BD.4, B.BD.2 and B.BD.3. A and B denote for the concrete strength M25 and B35, respectively.Table-5 shows the dimension and notation of bubble-deck samples. With the observation that only the sample A.BD.4 has been provided the links and other samples do not have links. All samples have 1900mm length, 800mm width and 230mm overall thickness. All details are shown in Figure-4.

Table-5. Shape and dimension of plastic balls [15].

Slab	Concrete strength		Dimension1900x800x230mm		
			BDØ186 (nolink)	BD Ø240 − 180 (nolink)	BD Ø240 − 180 (havelink)
Notation	A	B25	A.BD.2	A.BD.3	A.BD.4
	B	B35	B.BD.2	B.BD.3	-

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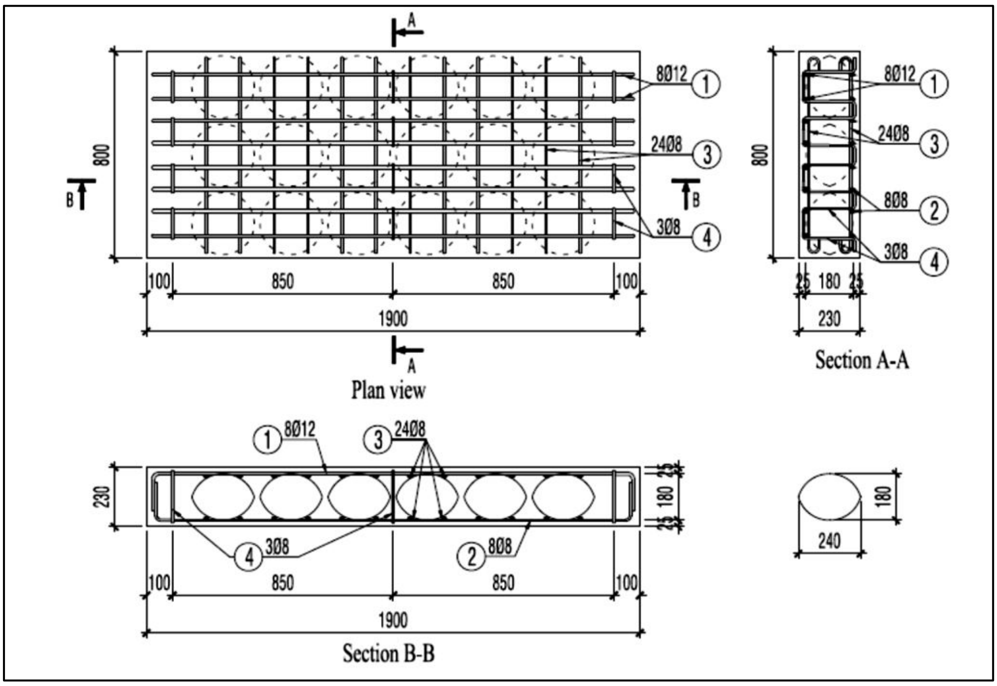

Figure-4. Shape and dimension of plastic balls [15].

The experimental program was carried out on one-way simply supported bubble-deck slab samples. The applied force will be increased step by step at the center of slab until the cracks are found in the slabs and the failure modes are appeared. It can be concluded from test results that the loading capacity of bubble-deck using concrete grade B35 is higher from 3% to 8% compared with that of bubble-deck using concrete grade B25. By using the modified bubble-deck with hollow elliptical balls D240-180, the loading capacity is increased from 6% to 11% as compared to that of traditional bubble-deck with hollow spherical balls D186.

Olawale and Ayodele (2014)[16]casted twenty model samples for the experiment. The samples were divided in two groups. Each group includes ten samples; the first group consists of slabs with small dimension 900 mm x 300 mm (W1 for small waffle and S1 for small solid slab) and were supported on all four sides, while the second group with large dimensions 1353 mm × 430 mm (W2 for large waffle slab and S2 for large solid slab) but were simply supported on the two short sides. Overall depth of waffle slab was 50 mm with the slab portion having 15 mm depth, waffle slabs are assumed to have band beams along all four edges. The width of band beam was taken to be 30 mm and the width of ribs was 20 mm. W1 has its rib spacing on the long span to be 200 mm and 170 mm on the short span, while W2 has 260 mm and 135 mm rib spacing on the long and short span, respectively, the solid slab has an overall depth of 40 mm. The samples were subjected to axial loads using five samples per panel.

The major and minor ribs in the waffle slabs were reinforced using 1Ø6 mm. while the slab portion was provided by plain 2.5 mm wire mesh. For the solid slabs, the reinforcement was Ø6@100 mm c/c. The compressive strength of normal concrete at 28 days was 20 MPa and steel tensile stress of 250 MPa. The slabs were loaded using universal tensile machine. Deflections of the slabs were measured directly at an interval of 1 kN until the failure of the system. The average mean value of the five results for each of the samples was used. Load - mid span deflection relationship for both groups are illustrated in Figures5 and 6.

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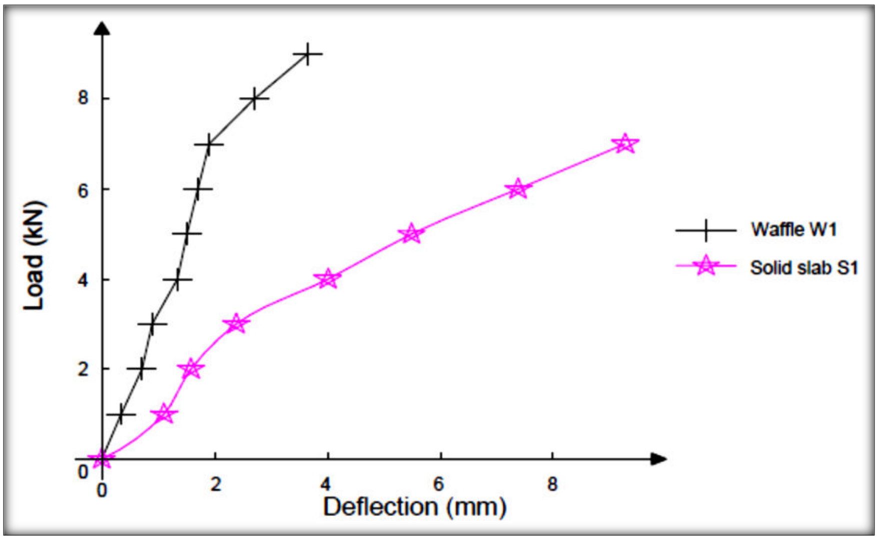

Figure-5. Load-mid span deflection relationship of samples S1 and W1 [16].

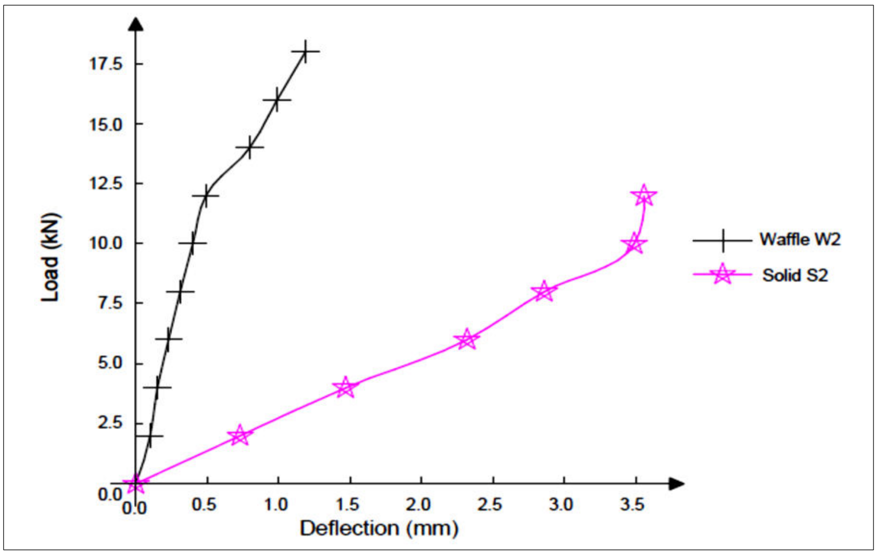

Figure-6. Load-mid span deflection relationship of samples S2 and W2 [16].

The results of first group for waffle W1 and solid S1 showed that the flexural strength of the waffle slab is higher than that of the solid slabs by 22%. With regard to the second group, waffle slab W2 was 33% higher than the solid slab S2 in flexural strength as given in Table6.It was observed that waffle slabs have flexural rigidities higher than solid slabs and this shows the advantages of waffle slabs compared to solid slabs in terms of the ability to support heavy loads over a long span without increasing the depth.

Table-6. Test Resultsof solid and waffle slabs [16].

Type of slab	**Bending moment** (kN.mm)	**Deflection at failure load** (mm)	**Crack width at failure load** (mm)
W1	1122	3.64	0.35
W2	5526	1.19	0.6
S1	872	9.28	0.75
S2	3684	3.56	1.20

de Oliveira, *et al* (2014)[17]carried out experimental investigation on eight one way ribbed slab panels to detect the contribution of slab portion in resisting the shear stresses. Rectangular (L1300) and square (L2000) panels have dimensions (1300 x2000 mm) and

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(2000x2000 mm) respectively. The details of panels are shown in Table-7.

Table-7. Details of tested slabs [17].

Slabs	$\mathbf{I_O(mm)}$	$\mathbf{h_f(mm)}$	**d**(mm)	$\mathbf{f_{ck}(MPa)}$	$\boldsymbol{\rho\%}$
L1300-30	610	30	277	35.2	1.1
L1300-50	610	50	277	35.2	1.1
L1300-80	610	80	277	35.2	1.1
L1300-100	610	100	277	35.2	1.1
L2000-30	960	30	277	35.2	1.1
L2000-50	960	50	277	35.2	1.1
L2000-80	960	80	277	35.2	1.1
L2000-100	960	100	277	35.2	1.1

I_O: Spacing between the Ribs. h_f:Thickness of flange.f_{ck}: Cube compressive strength d: Effective depth. ρ: Reinforcement ratio.

The overall depth of panels was 300mm with different thicknesses of flange. The width of ribs 80 mm and spaced of (I_O) 610 mm in group L1300 and 960 mm in group L2000. All slabs have the same longitudinal (flexural) reinforcement, consisting of $2\emptyset 12.5$ mmwith yield strain equal to 2.3%. The slabs were loaded under two line loads with (a/d) equal to 2.15.

The results of testing one-way ribbed slabs have shown that all slabs were controlled by shear failure (with and without yielding of flexural reinforcement) and the effective contribution for flange in resisting the shear stresses. The flexural reinforcement strains were proportional with the increase in the depth of flange which provides more ductility. No significant increase in ultimate load was recognized increasing the width of panels. The results of tested slabs are listed in Table-8.

Table-8. Ultimate load, mid-span deflection and failure mode [17].

Slab	**Ultimateload** $\mathbf{P_u}$ **(KN)**	**Mid − span deflection**$\boldsymbol{\Delta_u}$**(mm)**	**Failuremode**
L1300-30	200	1.81	shear without yield
L1300-50	210	3.56	shear without yield
L1300-80	290	7.38	shear with yield
L1300-100	360	11.32	shear with yield
L2000-30	160	9.54	shear without yield
L2000-50	220	12.69	shear with yield
L2000-80	330	14.08	shear with yield
L2000-100	370	15.41	shear with yield

According to ACI 318-2014, the ribbed slab provides the shear capacity by the cross-section of the ribs with modification factor equal to (1.1) Chapter 9- section 9.8.1.5. The increase rate of the ultimate load ranged from (40% - 56%) as result to the variation in the thickness of flange (30-100 mm) for both groups L1300 and L2000 respectively, ACI 318 provides very conservative shear capacity (144 KN) where no considerable for contribution the flange in shear capacity.

The use of wide beam ribbed slabs has become widely popular due to their economic benefits as shown in Figure-7.

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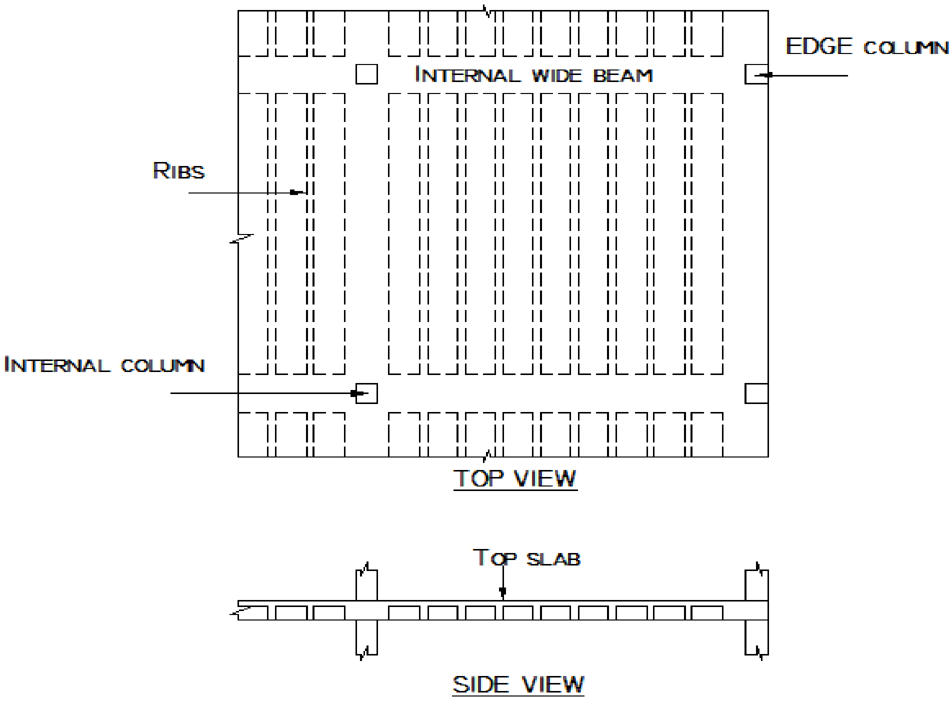

Figure-7. Ribbed slab with wide beam [17].

A wide beam ribbed slabs have main wide beams that are much wider than the supporting columns, distributed in the two orthogonal directions, and the ribs spanning between the beams in only one direction.

Punching shear is a type of failure of reinforced concrete slabs subjected to high localized forces. In the flat slab system, this occurs at column region. The failure is due to shear. At failure, a solid revolution of concrete (marked as ‘i’), which is the portion of concrete surrounded by the inclined shear cracks, separates normally from the slab leaving the rest of the slab (marked as ‘ii’) remaining rigid. The punching resistance of the slab is the sum of all the shear strength on the shear failure surface. Many researches were conducted to identify the solid slab behavior at columns. It is not clear how the items of codes related to the shear forces for the solid slab to wide beam which support the ribbed slab can be applied. When the wide beams within the ribbed slab systems are very wide, the punching failure surface could form within the full depth section Figure-8-B, but if the beams are narrow, the punching failure surface could pass through the reduced depth section, would lead to a lower punching shear capacity Figure-8-A.

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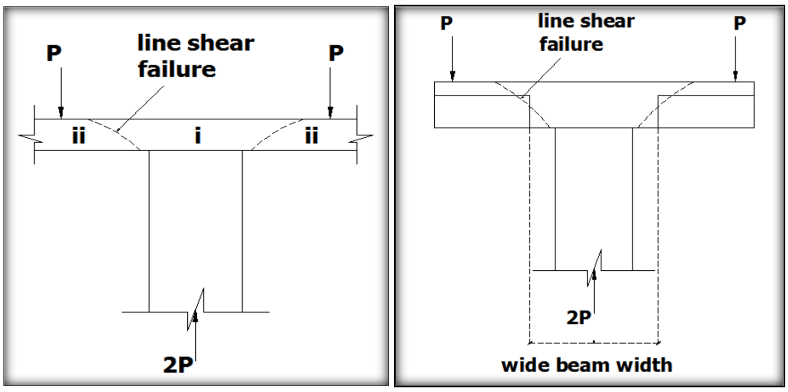
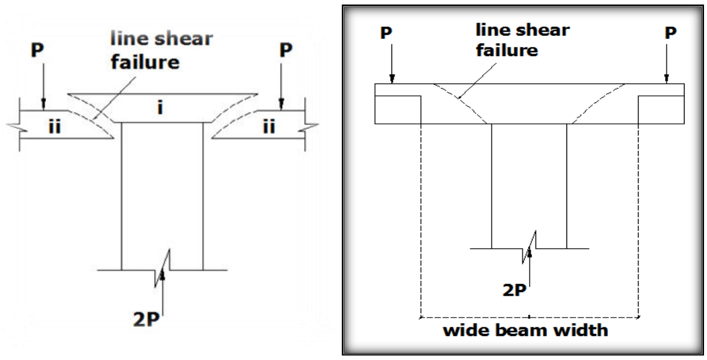


Formation of Shear Failure Wide Beams are Narrow



Punching Mechanism of SlabWide Beam is Very Wide
Formationof Shear Failure.

A**-** Solid slab B-Ribbed slab

Figure-8. Punching shear mechanism [17].

Allawi. N (2014) [18] carried out experimental tests on one-way voided slab to investigate the structural behavior of the reinforced concrete slabs containing cavities. The cavities were filled with styropor as insulation material and placed at the middle zone of the slab thickness between the tension cord (lower zone) and the compression cord (upper zone). The weight of specimens was reduced and provided by insulation properties as compared to the solid slab (reference slab). Fourteen reinforced concrete slab specimens were casted with 750mm length and 350mm width (the thickness of slab and cavities of all specimens 40mm and 12mm, respectively) and divided into two groups. The first group (S) includes nine specimens which contain separated embedded styropor pieces at the middle of slab thickness with different sizes and shapes. And the second group (T) consists of four specimens with full layer of styropor including concrete ribs. Figure-9 shows the number and arrangement styropors within the specimens.

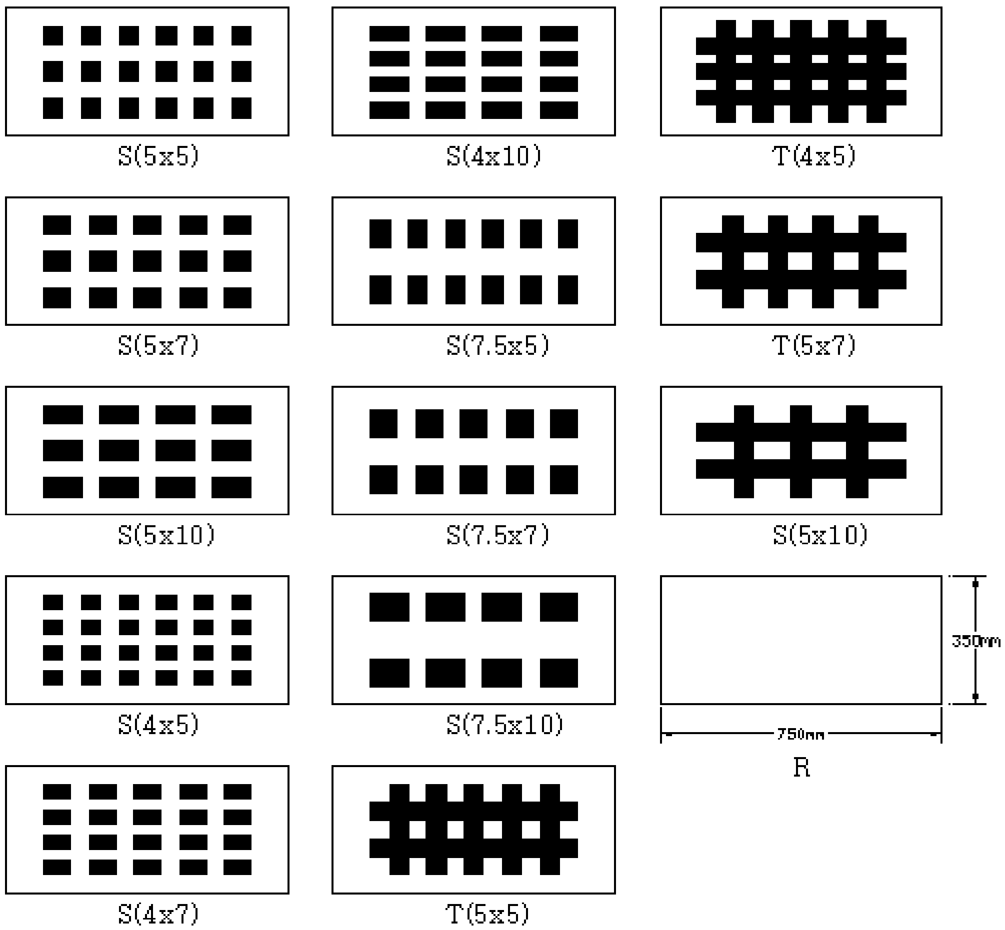


Figure-9. Number and arrangement styropors within the specimens [18].

Scaling down factor of about (1/6) was chosen. The compressive strength (f_{cu}) of 30MPais used for all specimens. Smooth reinforcing steel bars of average diameter 2.5mm were used with yield strength value of bars was 764MPa. Steel meshes were made by using steel bars of 2.5mm diameter in each direction. In the long direction (main reinforcement), the reinforcement was 10-Ø2.5mm placed at 35mm center to center c/c of the bars, except the first bar which is placed at 17.5mm from the edge of slab specimen. In the short direction (shrinkage and thermal reinforcement), 11-Ø2.5mm were spaced at 70mm c/c, except the first bar which is placed at 25mm from the edge of slab specimen. For all slab specimens with cavities the nominal strength was the same since the cavities are located at depth of 14mm from the top surface, while Whitney rectangular compression stress block depth is located at depth of 5.03mm for this reason the existence of holes inside the cross section does not affect the nominal strength of the section according to ACI code if these holes are not common with the compression block. The test results showed that the best percentage of weight reduction in cross sectional area according to this study was 13.7% since it gave an ultimate load capacity equaling 96.8% of the reference slab.

The results also showed thatthe more concrete ribs used inside the cross sectional area provide higher ultimate bending capacity and the number of transverse inner ribs has effect on the load capacity of the slabs. Decreasing the number of transverse inner ribs caused decreasing load capacity. The best number of transverse inner ribs was 5 of 50mm since it gives bending strength 98% of Reference slab. Table-9 shows the results for all specimens.

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Table-9. Compression between the ultimate moment and the nominal strength [18].

specimen number	Ultimate load P_u (kN)	P_u /2 (kN)	Ultimate actual moment (kN. mm)	Theoretical moment (nominal strength) (kN.mm)	Ultimate actual moment/theoretical moment
R	11.5	5.75	1455.2	1179.5	1.23
S(5x5)	11.3	5.65	1427.9	1179.5	1.21
S(5x7)	11.25	5.625	1421.7	1179.5	1.20
S(5x10)	10.85	5.425	1371.7	1179.5	1.16
S(4x5)	11.35	5.675	1434.1	1179.5	1.21
S(4x7)	11.25	5.625	1421.6	1179.5	1.20
S(4x10)	11	5.5	1390.3	1179.5	1.17
S(7.5x5)	10.75	5.375	1359.2	1179.5	1.15
S(7.5x7)	10.5	5.25	1327.9	1179.5	1.12
S(7.5x10)	10.25	5.125	1296.7	1179.5	1.10
T(5x10)	10.5	5.25	1326.4	1179.5	1.12
T(5x7)	10.45	5.225	1320.2	1179.5	1.12
T(5x5)	10.15	5.075	1282.7	1179.5	1.08
T(4x5)	9.75	4.875	1232.7	1179.5	1.04

Abed, 2016[19]carried out an experimental investigation on the flexural behavior of reinforced concrete slabs having longitudinal hollow- cores with different sizes and under different loading conditions by changing the ratios of shear span distance to effective depth ratio (a/d). The study includes investigation of eight moderately thick reinforced concrete slabs with and without longitudinal hollow cores. The specimens of the slab models have (2.05m) length, (0.6m) width and (25cm) thickness. These slab models were casted and treated at the same laboratory conditions and then tested by using flexural testing machine at the laboratory of the civil engineering department, Al- Nahrain University under two-point load and rested on simple supports. Three solid slab models were tested under the prescribed loading with three shear span-depth ratios (2, 2.5 and 3). Three hollow core slabs with three sizes of circular cores with diameter (75,100 and 150mm) were tested under shear span-depth ratio (a/d) equal (2.5). Two additional slab models with hollow cores (dia. =150mm) were tested under the prescribed loading with two ratios of (a/d) (2 and 3) to compare with solid types. Load-deflection values were recorded with each increment of load up to failure. First cracking load and ultimate load (failure load) during cracks propagation were recorded with their deflections at mid span and under applied two-point load. The test results showed that the ultimate strength of solid slabs decreased by about (21% and 33%) and the mid span deflection decreased by (13% and 48.5%) with increasing shear span ratio (a/d) from (2) to (2.5 and 3) respectively. The presence of circular hollow cores reduced the ultimate strength by (5.49%, 15.7% and 20.6%) with using circular hollow cores having diameter (75,100 and 150mm) respectively. When the shear span to effective depth ratio (a/d) increased from (2) to (2.5 and 3) respectively, the ultimate strength of hollow-core slabs decreased by (31% and 45%) with increasing in the deflections by (24.8% and 6.8%) respectively.

Daraj, (2017) [20] conducted experimental studyto explain the general behavior of hollow block slab when the reduction in weight range forms (23.3%-29.1 %).The main objective of experiments was to find the optimum ratio of weight reduction by removing the inefficient parts of the concrete under different (a/d) ratios. Also improving the failure mode using minimum shear reinforcement and solid slab portion or variable cross section (tapered section) along critical shear region.The test results showed that the best percentage of reduction in cross sectional area according to that study was ranging from 29%-35%. Under loading case A (a/d = 3.125) and reduction weight 29.1%, the hollow slab (HS) strengthened by minimum shear reinforcement gave an increase in ultimate load capacity equaling 8.6% compared to RS slab and 98% compared to RS slab with depth 85mm. At the same reduction weight and loading case B (a/d = 2), the increase in ultimate load capacity was 5.7% compared to reference slab (RS) slab. The HS slab provided by solid slab portion under reduction weight (29.1%) exhibited an increase in ultimate load capacity 7.4% compared to reference slab (RS) under loading case A (a/d = 3.125) and a decrease equal to 6.4% under the loading case B (a/d = 2), The wide ribs in HS slabs showed better results than those which have narrow ribs at the same cross sectional area.

Abdul Al-Aziz, (2017), [21] carried out the experimental study includes testing seven reinforced

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concrete slabs under two vertical line loads. Those slab specimens were casted treated and tested at the same laboratory condition and machinery. The dimensions of slab specimens were (1.1m) length, (0.6m) width and (12cm) thickness. The specimens are divided into two LWA solid slabs with varying (a/d) ratio; one normal aggregate solid slab with (a/d) equal (2.9); two LWA hollow circular core slabs (HCCS) with core diameter (50mm) with varying (a/d) ratio; one LWA hollow square core slabs (HSCS) with core side length (50mm) with (a/d) equal (1.9) and one normal aggregate (HCCS) with core diameter (50mm) with (a/d) equal (2.9.

In that study, the maximum reduction in weight due to aggregate type was (19.28%). However, it was (17.365% and 13.64%) due to cross section (square and circular) hollow cores, respectively. The test results showed that the decrease of shear span to effective depth ratio for lightweight aggregate solid slab caused an increase in flexural ultimate strength by (29.06%) and increase in deflection value at ultimate load by (17.79%). The use of lightweight aggregate concrete in casting solid slabs gives a reduction in weight by (19.28%) for a constant (a/d=2.9). While in cracking and ultimate strengths by (16.37%) and (5%) respectively for constant (a/d=2.9).The use of lightweight aggregate concrete in casting hollow circular core slabs with 2.9 value for a/d ratio (reduction in weight 32.92%) decreases the cracking and ultimate loads by (12%) and (5.18%) respectively with respect to solid slab. Using lightweight aggregate concrete in casting hollow square core slabs with constant (a/d=1.9) (reduction in weight 36.64%) decreases the cracking and ultimate loads by (14.29%) and (27.70%) respectively compared to the solid slab of same aggregate type. The use of lightweight aggregate concrete in casting hollow circular core slab with (a/d=2.9) (reduction in weight by 32.92%) shows lesser cracking and ultimate loads of (21.43%) and (4.69%) then the same slab (reduction in weight 13.64%) with normal weight aggregate.

4. THEORETICAL STUDIES ON VOIDED SLABS

The following research was carried out on voided slabs using numerical studies.

Elliott *et al.* (1982) [22] presented finite element procedures for determining the plate bending stiffness of a circular voided slab. The results were compared with previous results which obtained from tests on model elastic voided plates. Preliminary investigations showed that a simplified plane-strain formulation could be used to determine transverse flexural stiffness. Torsional stiffness was obtained from an analysis using the stress-function formulation of the torsion problems. Tests were carried out on four epoxy resin model plates having different void sizes. Good agreement was obtained between the finite element and experimental results. For design purposes, charts were presented which enable the designer of a concrete voided slab, having a Poisson's ratio of 0.2 to determine the values of the plate bending stiffness required for a thin plate analysis of such a slab. It was also shown that simple calculations based upon replacing a circular void with an equivalent square void were often adequate for design purposes.

Abdel Rahman and Hinton(1986) [23] presented linear and nonlinear finite element analysis of reinforced concrete cellular slabs based on a slab-beam model. The voided slab was treated as an assembly of hollow plate elements representing the flanges with beam elements representing the webs. The adopted layer approach allows for variations in the material state across the thickness. Applications of this model were presented for linear elastic problems and also nonlinear applications including reinforced and pre-stressed concrete voided slabs. It was showed that the present slab-beam formulation was adequate in describing the behavior of composite slab-beam structures.

Stanton (1992) [24] presented a comprehensive analytical program, supported by test results, was conducted to determine the distribution of response in hollow-core slab floors subjected to concentrated point and line loads. An extensive computer generated parametric study was used to accurately analyze the responses. The results were then condensed to produce simple rules which design engineers can use for the analysis of hollow-core slab system. Fully worked numerical examples to demonstrate the use of the proposed analysis rules and compare them with the rules contained in the "Precast/Pre-stressed Concrete Institute (PCI) Manual for the Design of Hollow-Core Slabs.The research found that these proposed analysis rules reflect the actual behavior of the floor more closely than the rules presently in the PCI Manual for the Design of Hollow-Core slabs and show that the latter are too conservative in some cases and too liberal in others.

Yang (1994) [25] presented a design procedure to determine the capacity against shear failure of webs of pre-stressed hollow core slabs. The additional shear stresses in webs caused by pre-stress forces of strands were taken into account in this procedure. The proposed procedure was applied to model slabs and also to test specimens. The capability of the proposed procedure to predict the shear failure of web was demonstrated by a good agreement that was obtained between the results predicted by FEM and the results predicted by the suggested formula. Agreement is also good between the test observations and the results predicted by the suggested formula. It was concluded that the proposed procedure can more accurately predict the capacity against shear failure of web than the formulas suggested in design codes. The procedure is reasonable and general in nature, and it can be applied to different loading cases, pre-stress levels, types of cross section, material, and geometric parameters.

Pajari (1998) [26] proposed a simple calculation model for prediction of the shear resistance of pre-stressed hollow core slabs supported on beams. A simplified principal stress criterion for the web shear failure is introduced. The criterion is based on three stress components. The vertical shear stresses (τzy) and the axial stress (σy) are calculated as for slabs supported on nonflexible bearings. The transverse shear (τzx) stress is calculated by applying beam theory to a composite beam comprising the beam,

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joint concrete, a section of the slabs, and the optional topping. The width of the section is an experimental parameter that takes into account the degree of composite interaction between the slabs and the beam. It was assumed to be proportional to the span of the beam. Also presented the modifications needed when the beams are continuous, the floor was covered with reinforced concrete topping, or the ends of the hollow cores were filled with concrete.

Helén and Lundgren (2002) [27] presented calculation method for shear and torsion in hollow core slabs including stresses from various influences without considering deformations and compatibility. The softening of cracking concrete, or restraint at the boundaries, and is therefore most likely conservative. Finite element analyses were carried out for individual hollow core units, subjected to different combinations of shear and torsion. Pre-stressed hollow core units of two thicknesses, 200 mm and 400 mm, were tested both with and without eccentric loading. The analyses were made with various levels of detailing, using the finite element program DIANA 7.2.The slab was modelled with beam elements and concrete was modeled using non-linear fracture mechanics in a smeared rotating crack model. In general the finite element analyses of the slabs were able to capture the overall behavior, failure mode, crack pattern, and maximum obtained load, with a reasonably good agreement, though a very coarse mesh were used in the analyses. Especially for the centrically loaded specimens, the agreement was good. However, for the eccentrically loaded hollow core units the maximum load was overestimated. The reason for this is most likely that the torsional stiffness of the beam elements used in the model was too high. Some analyses aiming at a shear flexure failure were also carried out. The aim of that analysis was to search for a loading situation where a shear flexure failure would be critical. In some of the investigated load situations, shear flexure cracks appeared. Yet, due to stress locking in the analyses, large tensile stresses were transferred over the shear flexure cracks, and thus, it was concluded that the shear flexure failure could not be described in the type of model used. However, it was concluded that a shear flexure failure is rather unlikely to occur for slabs where standard pre-stress levels were used.

AL-Maleki (2006) [28] developed linear analysis using a program based on the proposed method which suggested by Stanton. Also a nonlinear three- dimensional finite element analysis have been used to predict the load-deflection behavior of prestressed concrete slabs with hollow core using ANSYS v.5.4 program modeling the steel and concrete by using nonlinear behavior of the materials. The modified Newton-Raphson method is used for the nonlinear solution algorithm for the equations of equilibrium. Three slabs with and without hollow cores were analyzed and a comparison was made with experimental load-deflection curves of these slabs. Parametric study was carried out to investigate the effects of voids such as compressive strength of concrete, amount of the steel reinforcement, shape, number and diameter of the voids. The results indicate that the square and rectangle shapes instead of circular reduce the ultimate capacity by 2.8% and 14.7% relative to circular voids shape respectively. It was observed also when using three or four voids instead of two voids can decrease the stiffness and the ultimate capacity of the slab by 3.9% and 2.2 respectively relative to hollow slabs with two voids.

Yang, *et al* (2007) [29] adopted the basic principle and method of super finite element for analysis of voided slabs structure (which consists of two-way voided slabs with paralleled circular tubes) so as to simplify calculation, because the analysis of such slabs by the regular finite element method is extremely time-consuming and sometimes could not work out solution if personal computer were used. Eight-node flat shell element with the effect of transversal shear deformation was selected as super element and 3-D eight- node isoparametric solid element as sub-element. Their displacement modes and displacement transition matrix were presented and the relevant computing program was compiled. Voided slabs cases with different spans, lengths ratios and boundary conditions were computed by using super FEM and regular FEM respectively. Comparison between results of deflection from two methods shows that the super finite element method is simple and practical.

Noorzaei *et al*. (2009) [30] presented the analysis, design and graphical features of the different types of precast industrialized slab systems in the form of a generalized computer code. The program is written in FORTRAN language environment and can run on any small PC. The computer code is a user-friendly program, with several options, covering different types of slab systems. The most commonly used slabs included in that study were the solid one way, solid two way, ribbed, voided and composite slabs. The program always starts with the minimum dimension of the slabs (as defined by the Industrialized Building System (IBS). The IBS requires building components and their dimensions to be standardized) and evaluates the most optimum sections, based on the least weight section which is represented by the sections with the smallest depth. After deciding the details of the final cross-section of the floor slab, this software equipped with graphic facilities can draw the structural members to be viewed on the computer terminal. The illustrated examples show that the results obtained using the developed computer code and from the manual calculation are in excellent agreement. Furthermore, the application of the program, on the voided slab, indicates that the program can select the optimal dimension of the slab and voids for minimum weight and, thus minimum cost of the slabs.

Chung, et al. (2010) [31]studied the optimal hollow sphere shapes a biaxial hollow slab with 8900mm length, 300mm width and 250mm thickness through numerical simulation using nonlinear finite element methods. The methods were executed to verify the effect of hollow shapes. The shapes of hollow spheres which studies were square, sphere, mushroom, ellipse, rect. donuts (30, 50) and round rect. (50, 70) shows in Figure-10.The nonlinear finite element program LUSAS was used for analysis and finding out the parameters of hollow

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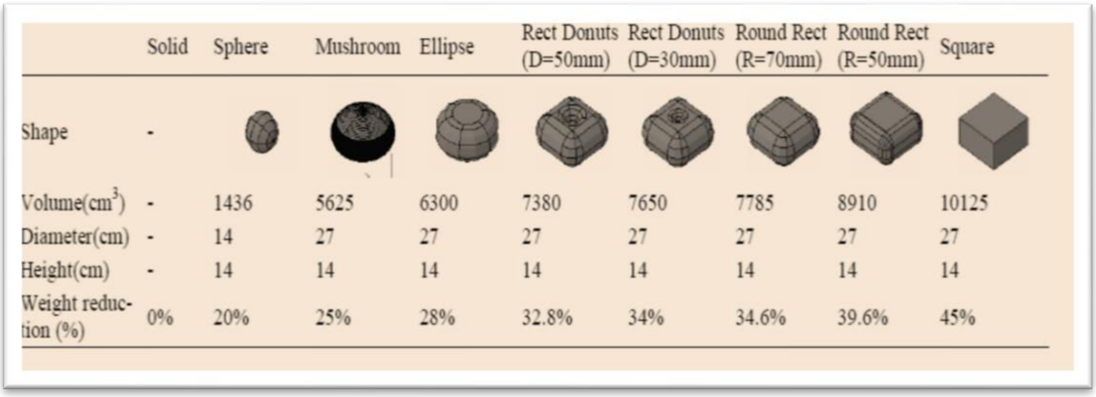
shapes such as typical shape, corner radius and hole diameter to find qualitative effects on hollow slab such as crack propagation or concentration. Three dimensional model was used for this case of bi-axial hollow in longitudinal and transverse direction. The optimal design after analysis of the hollow slab was rect. donut (50mm) and more than 99% of load resistance capacity and less than 72% deflection at design load which compared normal slab.



	Solid	Sphere	Mushroom	Ellipse	Rect Donuts (D=50mm)	Rect Donuts (D=30mm)	Round Rect (R=70mm)	Round Rect (R=50mm)	Square
Shape	-								
Volume(cm^3)	-	1436	5625	6300	7380	7650	7785	8910	10125
Diameter(cm)	-	14	27	27	27	27	27	27	27
Height(cm)	-	14	14	14	14	14	14	14	14
Weight reduction (%)	0%	20%	25%	28%	32.8%	34%	34.6%	39.6%	45%

Figure-10. Shape and dimension of hollow spherical shapes chung, park, *et al*, (2010). [31].

Mahdi (2011)[32] carried out nonlinear analysis of reinforced concrete hollow core slabs by finite element method using plate bending elements and beam elements to model the structure. The basic idea was to separate the hollow core slab into two main components. Hollow plates representing the upper and lower flanges and stiffening beams representing the vertical webs between the voids, Application of numerical analysis methods based on the finite element technique for solvingpractical tasks allows performing virtual testing of structures and exploring their behavior under load and other effects in different conditions taking into account elastic and plastic behavior of materials, appearance and development of splints, cracks and other damages (disintegration). A computer program from the book Hinton and Owen (finite element for plate and shell) was modified for analyzing various reinforced and pre-stressed concrete hollow core slabs and the finite element solutions were compared with the available experimental results to demonstrate the potential of the computational nonlinear model. By obtaining load-deflection response, several parametric studies were carried out to investigate the effects of some important finite element and material parameters on the behavior of reinforced and pre-stressed hollow core slabs. These studies included the effect of grade of concrete, amount of pre-stressing steel, presence of voids, void size, void shape and ultimate crushing strain of concrete. In general, good agreement between the finite element solutions and the available experimental results was obtained.

Amer, *et al.* (2012) [33] carried out a numerical analysis using finite element program (ANSYS v.12) to simulate the reinforced concrete slabs with spherical voids when subjected to five point load. Six slabs with length 1.0m, height (0.1m and 0.125m) with simply supported were modeled. Nonlinear materials behavior, as it relates to steel reinforcing bars and plain concrete, and linear behavior for steel plate was simulated using appropriate constitutive models. The results showed that the general behavior of the finite element models represented by the load-deflection curves at mid-span, ultimate load, load-maximum compressive strain curve of concrete, and crack patterns showed good agreement with the test data from experimental test. It was concluded that the finite element models can be used to carry out parametric study for the bubble deck slab specimens.

Saleh and Shahatha (2013) [34] proposed programmed mathematical techniques by Visual basic language for analysis, design and calculating the optimization of precast prestressed hollow core slab panels by adopting the modified Hooke-Jeevs method which is considered as a very suitable method especially for problems have many constraints. The formulizing of objective function according to required purpose discussed three parameters:

- Optimum weight,
- optimum cost and
- optimum live load

It was found that the average void percentage ratio regarding the optimum weight is about (50%) whereby the section tends to be in a shape where the voids become less than thickness and width take into consideration that the section was subjected to all the constraints (voids percentage tends to be much more than the regular case), as well as, it was found that the average of percentage of void ratio concerning with the optimum cost was about (41%). The research also adopted preparing the designable tables which are informative and practically for different kinds of hollow core slab sections. It was found from the tables of maximum live load that the deflection restricted the span length not less than (60%), furthermore the additional topping slab (5cm) thickness increased the span length about (16- 20) % for slabs with thickness (15-22) cm.

Brunesi, *et al*. (2014) [35] carried out experimental and finite element investigations on the shear strength capacity of precast pre- stressed concrete hollow core slabs. These slabs had thickness of (200-500) mm thick without transverse reinforcement. Through a detailed nonlinear finite element analysis, a matching experiments of test data collected by Pajari (1998) [26] was obtained .These members (49specimens) characterized with six nominal slab depths, five hollow shapes with circular and non-circular voids, different voids ratios, several pre-stressing steel strands arrangements and levels of initial pre- stress then comparative with traditional codes precipitations such as Euro Code2(EN1992-1-1) [36], ACI318R-05 [37] and Canadian Standards [38]. It was recognized that these members without transverse reinforcement can fail, due to web-shear cracking in the end regions, at loads less than those predicted by traditional Codes approaches (EC2, ACI). Hence, the shear strength capacity of these members without transverse reinforcement, characterized by intrinsic lack of ductility reserves, is evaluated through a campaign of detailed non-linear three-dimensional finite element analyses.

Al-Azzawi and Abed (2016)[36, 37] used the finite element method through ANSYS computer program was used to study the behavior of these reinforced concrete slabs. The Results of analysis showed good agreement with the experimental test results with difference of (4.71%-8.68%) in ultimate strength and (0.69%-9.31%) in mid span deflection**.**A parametric study have been carried out by using ANSYS program to investigate the effects of compressive strength of concrete, size and shape of core, type of applied load and effect of removing top steel reinforcement. The results indicate that the use of square shape for cores instead of circular shape reduces the ultimate capacity of slab by about 12%. The ultimate capacity of the slab increases with increasing the compressive strength of concrete by about 23.6% when the compressive strength of concrete increase from (38MPa) to (48MPa) but it decreases with removing the top steel reinforcement by about 28%

Al-Azzawi and Daraj (2017) [38] used nonlinear finite element analysis to investigate the behavior of hollow block slabs and to check the validity of the proposed numerical model using ANSYS14.5 package with the experimental model**.** Eleven models were tested with various parametric studies concerning boundary conditions, loads, material properties in order to study the behavior of modules. Good agreements with the experimental results in represent the eleven specimens. Also the researcher studied the behavior of hollow block slab when the reduction in weight more than 35%comparisons of their results for ultimate load and mid-span deflection with experimental reference slabs results. Specimen haven studied by the researcher deals with which model give high reduction in weight and stiffer than solid slab, after the verification and acceptability of the results between experimental work and the finite element method prepared appropriate parameters to work additional models under the finite element method. Four styropor block slab specimens investigated by ANSYS14.5 computer program with study parameters listed inTable-10and tested numerically under two monotonic line load (A) and (B). Figures 11-A and 11-B show the details of specimens HS7-A, HS7-B, HSC4-A and HSC4-B.the results showed The reduction in the compressive strength for models HSC4-A and HSC4-B by 27% led to decrease the strength capacity by 19% and 23% compared to models HS4-A and HS4-B as well as the characteristics of ductility.

Table-10. Study parameters proposed by ANSYS.

Specimens	$\mathbf{a/d}$	$\mathbf{R_w}$	**Unaixailcrushing Stress**$\mathbf{fc'(N/mm^2)}$
HS7-A	3.125	35%	37
HS7-B	2		37
HSC4-A	3.125	29.1%	27
HSC4-B	2		27

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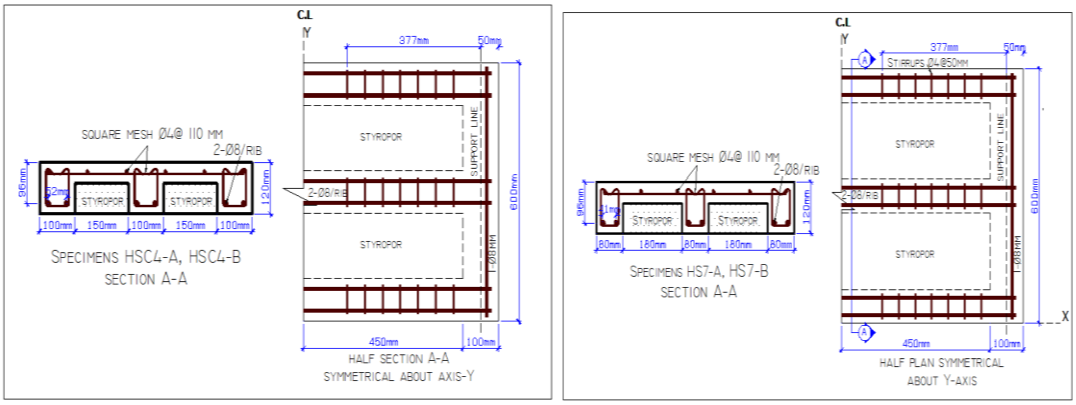

11- A11- B

Figure-11. Details of specimens HS7-A, HS7-B, HSC4-A and HSC4-B.

Al-Azzawi and Abdul Al-Aziz (2017) [39] studied the behavior of hollow core slabs numericallyusing the finite element method by using ANSYS computer program. The Results of analysis showed good agreement with the experimental test results of Abdul-Aziz with difference of (7.65%) in ultimate strength and (7.26%) in mid span deflection.A parametric study have been carried out by using ANSYS program to investigate the effects of number,diameter and area of cores,effect of load location,effect of adding top steel reinforcement, the effect of distance between applied load. The result show that ultimate load increased by (29.37%) when top reinforcement are used. The ultimate load decreases with increasing the number and diameter of cores by (8.9% and 16.28%) respectively.

5. CONCLUSIONS

Based on previous investigations, the behavior of voided slabs was simulated numerically and by carried out experiments on slab specimens with different void shapes, numbers geometry and reinforcement state. Voided slabs were recommended for roofing and flooring because of lesser self-weight, ease of construction and lighter supporting systems, foundations are required. The previous studies showed that

- It was observed that waffle slabs have flexural rigidities higher than solid slabs.
- The deflection was decreased as the voids diameters decrease, and also as the reinforcement increases.
- The decrease in the void-depth ratio improves the load distribution across the voided slabs.
- The test results showed that the best percentage of reduction in cross sectional area of voided slabs according to previous studies ranging from 30% to 40 %.
- The results indicated that the square and rectangle solids instead of circular reduce the ultimate capacity by 2.8% and 14.7% relative to circular voids shape respectively for the hollow core slabs.
- The increases in diameter of hollow core to double give an increase in torsional stiffness by 14%.
- The failure mode of hollow-core slabs changed from pure flexure mode to flexure-shear mode for slabs with ratio of depth to span greater than 200 mm.
- It is found that the decrease of shear span to effective depth ratio for lightweight aggregate solid slab causes an increase in cracking strength by and in ultimate strength. The mode of failure changes from flexural to shear flexural
- Using minimum shear reinforcement increased the shear capacity significantly.
- The best weight reduction percentages obtained in these studies for voided slab ranging from (19% to 35%).
- The results verified that full load distribution is attained up to the ultimate flexural capacity of the system based on a field test of hollow-core concrete slabs.

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