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MODE OF COLLAPSE OF SQUARE SINGLE PANEL REINFORCED CONCRETE SPACE- FRAMED STRUCTURES WITH RIGID BEAM-COLUMN JOINTS

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ABSTRACT
The behavior of the structural elements of a space-framed structure depends on their support conditions. These support conditions can be hinged or rigid for beam-column joints, while for slab these support conditions can be simply supported on walls that are not monolithically constructed together, simply supported on beams that are monolithically constructed together and, can be freely supported on one, two or three edges. This paper studies the mode of collapse of collapse of a single-paneled reinforced concrete space-framed structure with rigid beam-column joints. Five models were investigated for the interactive behavior of slabs, beams and columns. The models were loaded directly till collapse. The estimated and actual collapse loads of the five models were compared. The estimated collapse load for the slab was 35 kN/m². Also, the numerical estimate of the collapse load for the beam was 10.2kN/m (with an equivalent slab load of 40.8kN/m²), while the shear capacity at the beam-column joints was estimated to be 19.14 kN. The mode of collapse for all the five models was by shear failure at the beam-column joints at an average shear force of 7.13 kN as against the estimated shear capacities of 19.14 kN, showing that the existing formulae for predicting shear capacity of beam-column joints gave an overestimated value of joint shear capacity up to about 168%. It was found that the space framed models failed by shearing at the beam-column joints, and that the estimated shear capacity was greater than the shear force at collapse.

Keywords: *collapse load, moment of resistance, shear capacity, slab, beam and column joints.*

1. INTRODUCTION
The behavior of reinforced concrete space-framed structures largely depends on the boundary conditions of its interconnected structural elements. These boundary conditions dictate the mode of transmission of internal forces from one structural member to another. Also the strength characteristics of any structural member depend on its boundary condition with other structural members.

The boundary condition between beams and columns may be hinged or rigid, while that of slabs may be simply supported on supports such as walls which are not monolithically constructed together. Also, slabs can be supported on beams, which are monolithically constructed with the slabs. This presentation examines the effect of the rigid beam-column joint of a single-paneled reinforced concrete space-framed structure and is based on its statical strength and practical detailing characteristics and with its associated structural safety implications in constructions.

2. BACKGROUND OF STUDY
Figure 1a shows the detailing of a hinged joint of a reinforced concrete beam-column joint, while Figure 1b shows that of rigid beam-column joint. Hinged joints can only transfer shear and axial forces, while rigid joints can transfer shear force, axial force and bending moment.

The type of detailing provided at a joint, determines whether a joint is hinged or rigid. Inadequate structural detailing (i.e. total lack of transverse reinforcement in the joint region), deficiencies in the anchorage (use of plain round bars with end-hooks) and the absence of any capacity design principles can lead to the development of brittle failure mechanisms,

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particularly in exterior joints where additional sources of shear transfer within the joint region cannot develop after first diagonal crack. Local and global damage and failure mechanisms might be significantly affected by the consequent peculiar non-linear behaviour of the joint [1].

Various damage or failure modes are expected to occur in beam-column joints depending on whether the beam-column joint is an exterior or interior joint and, of the adopted structural details i.e. presence of transverse reinforcement in the joint; use of plain round or deformed bars and the type of bar anchorage provided [2]. As shown by experimental tests on beam-column joint specimens and a three storey frame system, the combination of strut action and of a concentrated compression force at the end-hook anchorage, due to slippage of the longitudinal beam bars, lead to the expulsion of a "concrete wedge", with rapid loss of bearing-load capacity [3, 4 and 5]. Furthermore, the premature slipping of the longitudinal bars introduces, through a concentrated compression force at the end-hook, an "equivalent" principal tensile stress state in the joint region, which anticipates the joint diagonal tensile cracking [1].

There are different methods of detailing a rigid joint; however, the type of detailing generally adopted in Nigeria is the type of detailing shown in Figure 1b. The mode of collapse of several buildings recorded in the Nigeria is by shear failure at the beam-column joint, which is sudden in nature, even when such buildings were designed to fail by gradual yielding of the tension reinforcement which is required to give adequate warning of the imminent failure of such buildings [6]. Therefore, the focus of this work is to investigate collapse mechanism of reinforced concrete space-framed structure with rigid beam-column joints.

A space-framed structure designed and detailed at the beam-column joints to transfer moment and shear forces is said to have fixed or rigid joints, while the one that is detailed with no capacity to transfer moment is said to be hinged jointed. The experimental study herein, contributes to the knowledge of the behavior of space-framed structures with the beam-column joints detailed to be rigid. A Better understanding of the behavior of reinforced concrete space-framed structures at failure will reduce the rate of collapse of buildings in the country when its adequate formulations are adhered to.

The collapse of most reinforced concrete structures is by shear failure at the beam-column joint and sudden in nature. Joint ACI-ASCE Committee 326 [7, 8 and 9] published a report regarding the design and behavior of beams failing due to shear and diagonal tension. To develop safe design recommendations, a database of 194 beam tests without shear reinforcement was compiled. The database consisted of 130 laboratory specimens tested under single and double point loads and 64 beams subjected to uniformly distributed loads. Based on those data, a design equation was formulated and is included in ACI 318-08 [10] and presented as equation (1):

$$V_c = \left(\sqrt{f_c^1} + 120\rho_w \frac{V_u}{M_u}\right)\frac{b_w}{7} \leq 0.3(f_c^1)b_w d \qquad (1)$$

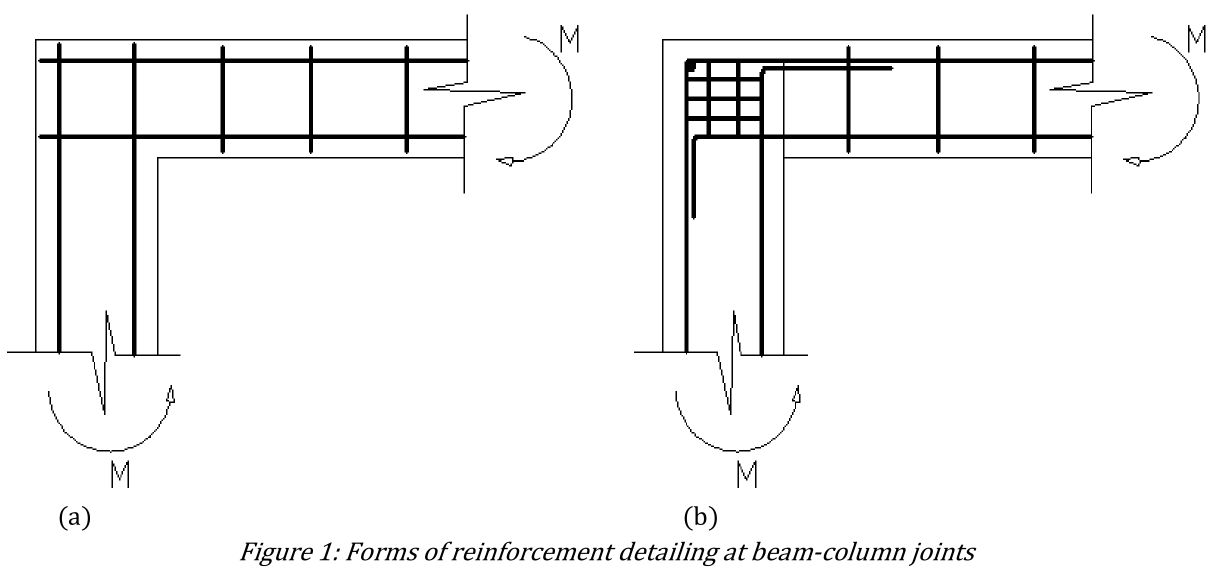

Figure 1: Forms of reinforcement detailing at beam-column joints

In (1), V_c is the nominal shear strength provided by concrete; f_c^1 is the specified compressive strength of concrete; ρ_w is the reinforcement ratio $\frac{A_s}{b_w d}$; V_u is the factored shear force at section; M_u is the factored moment at section; b_w is the web width; d is the effective depth of section; and A_s is the area of tension reinforcement.

By neglecting the term $\frac{V_u d}{M}$ in Equation (1) is a simplified but conservative version could be derived and presented as Equation (2).

$$V_c = \frac{1}{6}\left[\sqrt{f_c^1}\right] b_w d \qquad (2)$$

To include the effects of loading type and shear span to depth ratio into current code provisions, for members in which more than 1/3 of the factored shear at the critical section results from concentrated load located between 2d and 6d of the face of the support [11], proposes:

$$V_c = \frac{1}{12}\left[\sqrt{f_c^1}\right] b_w d \qquad (3)$$

Such a reduction in shear strength as indicated in Equation (3) will substantially reduce the number of tests that fall below code values.

According to Arslan [12], the nominal shear strength provided by concrete can be estimated using equation (4).

$$V_{cr} = V_{crt} + V_{crd} = 0.15(f_c^{0.5}) b_w d + 0.02(f_c^{0.65}) b_w d \qquad (4)$$

Where: V_{cr} is the cracking shear strength, V_{crt} is the diagonal tension cracking strength and V_{crd} is the dowel strength.

Based on the principal shear strength V_o carried in the compression zone, considering the influence of parameters; the slenderness ratio (a/d) and size effect (1/d), Arslan [13], expresses the diagonal cracking strength of RC slender beams without stirrups as given in Equation (5).

$$V_c = \left[0.2 f_c^{\frac{2}{3}}\left(\frac{c}{d}\right)\left(1 + 0.032 f_c^{\frac{1}{6}}\right)\left(\frac{4}{a/d}\right)^{0.15}\left(\frac{400}{d}\right)\right] b_w d \qquad (5)$$

Where c is the depth of the neutral axis.

Other existing shear strength models for slender beams without stirrups, [14, 15, and 16]are presented in Table 1.

The ACI 318-08 [10] design shear strength is a simple superposition of transverse reinforcement and concrete strength. The design strength is independent of whether flexural yield has occurred prior to shear failure. For members, design shear strength is calculated using equation (6).

$$V_n = V_c + V_s = \frac{\sqrt{f_c}}{6} b_w + \frac{A_w f_y d}{s} \qquad (6)$$

In (6), V_c is the contribution of concrete to shear strength; V_s is the contribution of shear reinforcement to shear strength; f_c is the compressive strength of concrete; A_w is the area of shear reinforcement within a distance s and f_y is the shear reinforcement yield strength. The contribution of shear reinforcement is derived from basic equilibrium considerations on a 45-degree truss model with constant shear reinforcement spacing and an effective depth.

In their work, Arslan and Polat [17], show that there exists a significant amount of contribution of concrete to the shear strength (18 - 69%), however, they noted that further experiments should be conducted with a wider range of shear reinforcement ratio, shear span-to-depth ratio, concrete strength and various loading schemes in order to obtain more reliable assessments.

Table 1: Some of the existing shear strength models for slender beams without stirrups

Investigator	Shear strength models
Kim and Park (1996)	$V_u = \left[3.5 f_c^{\alpha/3}\rho^{3/8}\left(0.4 + \frac{d}{a}\right)\left(\frac{1}{\sqrt{1+0.008d}} + 0.18\right)\right] b_w d$ $\alpha = 2 - \left(\frac{a}{d}\right)/3$ for $1.0 \le \frac{a}{d} < 3.0$; $\alpha = 1$ for $\frac{a}{d} \ge 3.0$
Rebeiz (1999)	$V_c = \left[0.4 + \sqrt{f_c \frac{a}{d}(2.7 - 0.4A_d)}\right] b_w d$ $A_d = \frac{a}{d}$ for $\left(\frac{a}{d}\right) < 2.5$ and $A_d = 2.5$ for $\left(a/d\right) \ge 2.5$
Khuntia and Stojadinovic (2001)	$V_c = \left[0.54\sqrt[3]{\rho\left(f_c \frac{Vd}{M_u}\right)^{0.5}}\right] b_w d$; $\frac{M_u}{Vd} = \frac{a}{d} - 1$

Since the mid-1980s, there is an increasing amount of experimental evidence showing that the underlying concepts of the provisions of current codes (for example, BS 8110-I [18] and ACI 318-08 [10]) for shear in particular and to a certain extent for flexural design of reinforced concrete (RC) structures are in conflict with fundamental properties of concrete at both the material and structural levels [19].

From previous research work [20] on five square single panel reinforced concrete space-framed structures with beam-column joints hinged, average shear capacity of the five models was 4.08 kN. Using design formulae in the BS 8110-1 [18] and, considering the dowel action of tension reinforcement, the predicted shear capacity of the models was 15.11 kN. However, neglecting the dowel action of tension reinforcement and, considering the resistance capacity of the concrete alone, the predicted shear capacity of the models was 5.7 kN. This value of 5.7 kN of shear capacity is more realistic when compared with the actual shear capacity of 4.08 kN, therefore confirming the assertion [19], that dowel action of the reinforcing steel has no part to play in the shear resistance.

Based on the results of Olanitori *et al* [20], it was suggested that in order to reduce the failure of reinforced concrete space-framed structures with hinged beam-column joints, Equation (7) can be used for the prediction of the shear capacity.

$$V = \lambda_c V_c \qquad (7)$$

Here λ_c is the concrete shear capacity factor, V is the shear capacity of the space frame and, V_c is the shear capacity due to concrete.

When the compressive strength of concrete, f_c, is less than or equal to $10 N/mm^2$, the concrete shear capacity, λ_c, should be taken as 0.7.

Also, Olanitori and Afolayan [21], in their investigative work on the causes of the collapse of a building which was detailed and constructed as rigid joints in Nigeria, noted that the actual shear capacity of the building was 18.88 kN, while the predicted shear capacity was 64.4 kN. This wide variance of about 241% between the actual and predicted shear capacities calls for more research into the behavior of reinforced concrete space-framed structures.

3. MATERIALS AND METHODS

The materials used for this work include five square, single panel, reinforced concrete space-framed models, constructed from micro-concrete (using sand from borrowed pit, which is the most popular source of sand in Akure metropolis of Ondo State), loading box, laterite as the loading material, and dial gauges. Figure 2a shows the Schematic figure of square RC Space Framed Model while, Figure 2b shows the schematic figure of square RC Space Framed Model with the Loading Box.

The loading box which measured 1m x 1m x 3.0 m was placed on top of each model, and three dial gauges were placed at the centre of the slab and mid-span of two adjacent beams. Manually, known weight of laterite using headpan was poured into the loading box and readings of the gauges taken at every 1.84 kN load of laterite. The process continued until collapse occurred. The collapse load of each of the square space framed models was estimated by calculating the moment of resistance of the slab and the beam. The mode of collapse was noted and the actual collapse load compared with the estimated or numerical value.

4. ANALYSIS, RESULTS AND DISCUSSION

4.1 Analysis

4.1.1 Standard Collapse Mechanism and Collapse Loads

Moy [22] has shown the four standard cases of collapse mechanisms for slabs with edge beams. The type of collapse mechanism for a particular statical or structural slab depends on the relative moments of resistance of the beams and the slab. The observations from the experiments are as noted herein in collapse mechanisms shown in Figure 2 (a) – (d).

Collapse mechanism (a): The edge beams are rigid, such that the diagonal mechanism forms in the slab. Only positive yield lines are required because the beams can rotate after torsion failure in the corners.

For this type of collapse mechanism, the collapse load can be estimated using Equation (8):

$$n = \frac{24M}{L^2} \qquad (8)$$

Collapse mechanism (b): This type of mechanism depends on $\frac{M_b}{ML}$, which is a measure of the relative magnitudes of the moments of resistance of the edge beams and slab. The collapse load for this type of mechanism can be estimated using Equation (9):

$$n = \frac{8M}{L^2}\left(1 + \frac{2M_b}{ML}\right) \qquad (9)$$

Collapse mechanism (c): Also this type of mechanism depends on $\frac{M_b}{ML}$, which is a measure of the relative magnitudes of the moments of resistance of the edge beams and slab. The collapse load for this type of mechanism can be estimated as for collapse mechanism (b).

Collapse mechanism (d): This type of mechanism can occur when one of the edge beams is weaker than the others, and collapse load can be estimated using Equation (10).

$$n = \frac{8M}{L}\frac{\left(1+\frac{M_b}{ML}+\frac{L}{4X}\right)}{\left(L-\frac{X}{3}\right)} \qquad (10)$$

In Equation (8) to Equation (10), M is the moment of resistance of the slab; L is the length of the slab and M_b is the moment of resistance of beam. Figure 2 indicates some standard collapse mechanisms for reinforced concrete slabs.

4.1.2 Structural Design of Models

The space-framed models were designed in accordance with the requirements of BS8110 [18]. The dimensions of the models are:

Slab: 1000mm x 1000mm x 50mm thick.

Beam: 75mm x 100mm.

Column: 75mm x 75mm. Column height = 1000mm.

After the design, the following areas of reinforcement were incorporated as determined from conventional reinforced concrete design practice.

Slab: A_{sreq} = 21.2mm^2; A_{sprov} = 10R6 bars at 100mmc/c, area = 283mm^2, both ways.

A_{smin} =65mm^2< A_{sprov} = 283mm^2<A_{smax} = 2000mm^2.

Deflection check was satisfactory.

Beam: A_{sreq} = 9.6mm^2; Asp_{rov} = 2R6 bars, area = 56.6mm^2.

A_{sreq}< A_{sprov}< A_{smax}.

Deflection check was satisfactory.

Shear Reinforcement for Beam: R6 bars at 100mm centres.

Column: A_{sreq} = 22.5 mm^2 ; Asprov = 4R6 bars, area = 113.2 mm^2

A_{sreq}< A_{sprov}< A_{smax}.

4.1.3 Moment and Shear Resistances

(a) Moment of resistance of slab

The moment of resistance of the slab was estimated using the stress distribution in Figure 3. For the equilibrium of horizontal forces as given in Equation (11):

$$F_{cc} = F_{st} \qquad (11)$$

$0.45f_{cu}xbxs = 0.87f_yA_s$Where s = 0.9x, and x is the depth of neutral axis. Hence

$0.45\ x\ 7\ x\ 100\ x\ s = 0.87\ x\ 250\ x\ 283$ Thus

$$s = \frac{0.87\ x\ 250\ x\ 283}{0.45\ x\ 7\ x\ 1000} = 19.5mm;\ \text{But}x\ = \frac{s}{0.9} = \frac{19.5}{0.9} = 21.7mm;$$

Hence, 21.7mm < 0.615d = 0.615 x 45 = 27 .7mm.

Therefore the tension steel has yielded with a slab moment of resistance,

$$M_{RS} = 0.87f_yA_s = 0.87\ \times\ 250\times 283\times\left(45-\frac{19.5}{2}\right)\times 10^{-6} = 1.9\ kNm/m$$

Using equations 14 and 15 of BS8110-1[1985] the collapse load can be obtained as in Equation (12):

$$m = \alpha n l_x^2 \qquad (12)$$

Where: $m = M_{RS} = 1.9kNm$; l_x = length of the short span of the slab = 1000mm

n is the ultimate or estimated collapse load in kN/m^2.

From Table 3.14 of BS8110-1[18], α = 0.055, and consequently the estimated collapse load for the slab n, becomes 35kN/m^2.

(b) Moment of resistance of beam

Considering the stress distribution shown in Figure 4, equilibrium of the horizontal forces implies that

$F_{cc} = F_{st}$ That is 0.45 f_{cu} b S = 0.87f_y A_s. Which results into s = 52.1mm and subsequently x = 58mm. It is now obvious that x < 0.615d and that the moment of resistance of the section can now be estimated from

$$M = F_{st}\ Z= 0.87\ f_y\ A_s\ Z \qquad (13)$$

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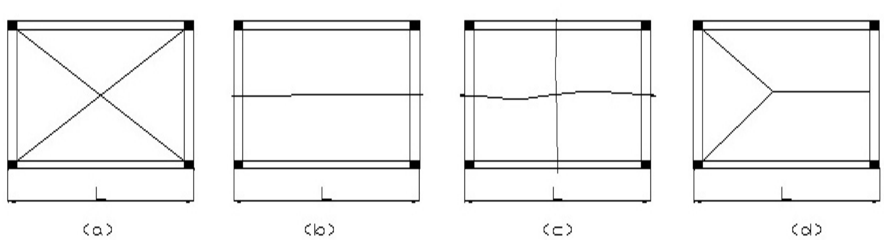

Figure 2: Standard collapse mechanisms for square slabs [22]

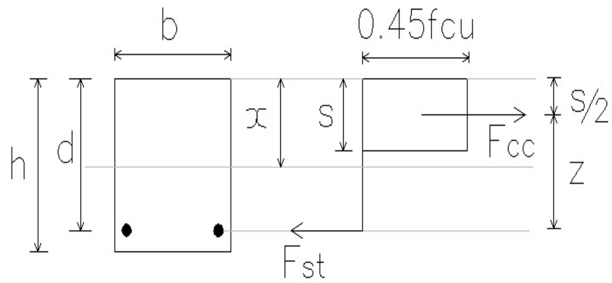

Figure 3: Stress distribution in a singly reinforced concrete section

In (13), F_{st} is the resultant tensile force in the tension reinforcement and Z is the lever-arm; A_s is the area of reinforcement provided. A_s = 56.6mm^2 and,f_y is the characteristic strength of steel and, f_y = 250N/mm^2 for mild steel.

Substituting for A_s, f_y and Z in equation (9), we have, M = 0.85kNm.

Since the beam-column joint is designed and detailed as fixed joint, the maximum bending moment at the mid-span is M = wl^2/12.

Substituting for all the relevant parameters, w = 10.2 kN/m.

The ultimate load can be estimated using

$$w = \frac{1}{4} n l_x \quad \text{Equation} \qquad (14)$$

Where

w is the estimated collapse load in kN/m; n is the estimated collapse load in kN/m^2, and l_x is the length of the short span beam. Substituting for w and l, we have n = 40.8 kN/m^2 as the estimated equivalent slab collapse load on beam

(c) Shear capacity of frame

The shear capacity V of a section for a given stirrup size and spacing can be estimated from Equation (15):

$$V = \left(\frac{A_{sv}}{S_v} 0.95 f_{yv} + b\vartheta_c\right) d \qquad (15)$$

Here, A_{sv} is the area of shear reinforcement provided S_v is spacing of shear reinforcement which equals the effective depth of the beam, f_{yv} is the characteristic strength of shear reinforcement, b is the beam width, v_c is the ultimate shear stress that can be resisted by concrete, and d is the effective depth of the beam.

Equation (15) can be rewritten in another form as expressed in Equation (16):

$$V = V_s + V_c \qquad (16)$$

Where V_s is the shear capacity due to stirrups, and V_c is the shear capacity due to concrete.

So that

$$V_s = \left(\frac{A_{sv}}{S_v} 0.95 f_{yv}\right) d \text{ and } V_c = b\vartheta_c d$$

Where: V_s is the shear capacity due to stirrups. V_c is the shear capacity due to concrete. ν_c is the ultimate shear stress of concrete.

The ultimate shear stress of concrete ν_c, is a function of the effective depth of the beam and the percentage area of tensile reinforcement. Thus, the percentage area of tensile reinforcement is:

100A_s/bd = (100 x 56.6)/ (75 x 95) = 0.79%. Therefore from Table 3.8 BS 8110 – 1[18]

ν_c = 0.8, so that V_s = 13.44kN and V_c = 5.7kN, given V = V_s + V_c = 13.44 + 5.7 = 19.14 kN

Shear stress ν = (19.14 x 10^3)/ (75 x 95) = 2.7N/mm^2> 0.8√7 = 2.12N/mm^2, hence the concrete at the compression zone would have started crushing at the attainment of a shear force equals 19.14kN. For the models, the expected shear capacity without the crushing of concrete in the compression zone is

V = 75 x 95 x 2.12 x 10^{-3} = 15.11kN.

4.2 Results and Discussion

4.2.1 Mode of collapse

A space-framed structure is said to be rigid at the beam-column joint if the beam reinforcements are bent to form hooks attached to the column reinforcements. The frame collapsed due to shear failure at the beam-column joints without the slipping of the beam reinforcements over the column. Because the beam reinforcements did not slip over the column reinforcements, this allows the beam longitudinal reinforcements to participate in the resistance of shear forces. Plate 1 shows the crack pattern under the slab, Plate 2 shows the crack at the mid-span of the beam and Plate 3 shows the crack at the beam-column joint of the space-framed reinforced concrete experimental model number B.

In Equations (5) to (7) the collapse load depends on M_b/ML which is a measure of the relative magnitudes of the moments of resistance of the edge beams and slab. Moy [22] shows that when M_b/ML > 1 slab collapse mechanism of Figure 2a is critical because the beams are sufficiently strong to resist collapse and, when M_b/ML = 1, each mechanism has the same collapse load and is equally likely to occur. However, from sections 4.1.3 (a) and (b), moment of resistance of slab M_{RS} = 1.9 kNm/m and that of beam M_{RB} = 0.85 kNm, from here, we have M_b/ML = 0.85/1.95 = 0.44 < 1 (since L = 1m). The crack pattern under the slab as shown in Plate 1 is same as that of Figure 2b; hence one can conclude that if ratio M_b/ML < 1, then collapse mechanism of Figure 2b is critical.

Since crack pattern under the slab is same as that of Figure 2b; the slab collapse load can be estimated using Equation (9).

Plate1: Crack at the bottom of space-framedslab (model B)

Therefore substituting for M = 1.9KNm, M_b = 0.85kNm and L = 1m, the slab collapse load n= 28.8kN/m^2.

The propagation of cracks started at the beam mid-span, because moment at the mid-span $M_{ms} = \frac{wl^2}{3}$is greater than the support moment $M_s = \frac{wl^2}{12}$.Immediately after the propagation of cracks at the mid-span, the propagation of crack at the supports started. The crack at the beam-column joint starts at the face of the column which is the support, from the top which is in tension due to the fixed joint, and bends almost at 45^0 to the column. This crack pattern at the support is against the general belief that cracks should develop at a distance (equal to the effective depth of the beam) from the face of support. The crack at the mid-span of the beam was due the flexural action of the beam while the crack at the support was due to combined flexural and shear actions of the beam.

Plate 2: Crack at mid-span of space-framed beam (model B)

Plate3: Crack at the beam-column joint of space-framed (model B)

The crack pattern of the beam compares favourably with that of the experimental works carried out on simply supported beam, loaded with uniformly distributed load [11]. For space-framed beam, the propagation of the crack started from the top which is in tension, while for the simply supported beam (Figure 4), the propagation of the cracks started from the bottom which is tension.

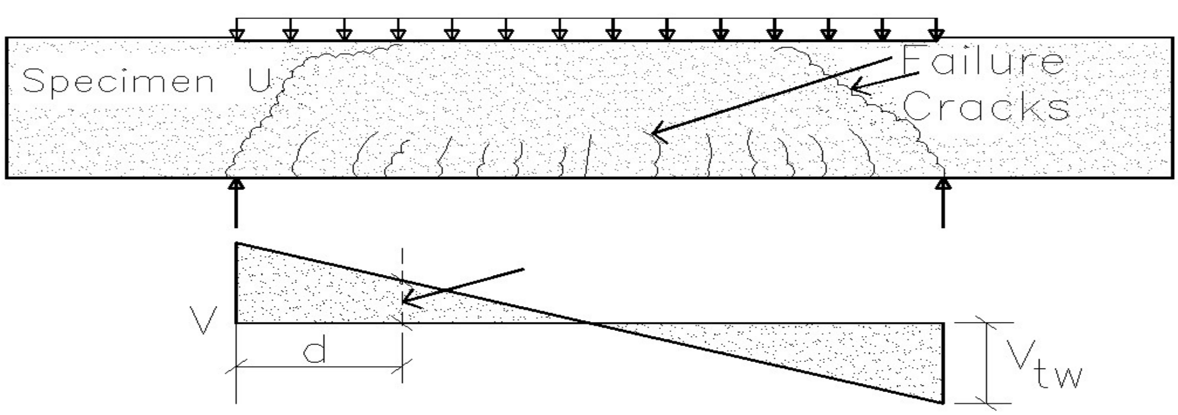

Figure 4: Failure conditions of simply supported beam with uniformly distributed load [11]

4.2.2 Shear capacity

Table 2 shows the actual shear capacities of the five models, from which the average experimental shear capacity is 7.13kN.The average shear capacity of 7.13 kN for the space-framed models with beam-column joint fixed is less than estimated shear capacity (using equation (16)) and is given as:

$V = V_s + V_c = 13.44 + 5.7 = 19.14$ kN.

However, the average experimental shear capacity of the beam section V = 7.13 kN, is greater than the estimated shear capacity due to the concrete V_C which is 5.7 kN. This shows that the tension and shear reinforcements contributed to the shear resistance of the beam-column joints.

Let

$V = \lambda_s V_s + \lambda_c V_c$, where using Equation(7)

$V_{CR} = \lambda_c V_c = 0.7V_c$

Equating V to the average shear capacity of 7.13kN, we have:

$V = \lambda_s V_s + \lambda_c V_c = \lambda_s \text{x}13.44 + 0.7\text{x}5.7 = 7.13\text{kN}$.

Therefore $\lambda_s = 0.23$.

Hence the proposed new formula takes the form of Equation (17):

$$V = \lambda_s V_s + \lambda_c V_c = 0.23V_s + 0.7V_c \quad (17)$$

Where V_{CR} is the reduced shear capacity of concrete, λ_c is the concrete shear capacity factor and

λ_s is the tension reinforcement and stirrups shear capacity factor.

Equation (17) above gives a value that corresponds to characteristic shear capacity of space-frames with rigid beam column joints.

The average shear capacity of the space-frame can be compared with that of ACI 318-08 [10] using Equation (18) and, Brown *et al* [11] using Equation (19). The two equations are stated below:

$$V_{ACI} = V_s + V_c = \frac{A_{sv}}{S_v} 0.95 f_{yv} d + \frac{1}{6}\sqrt{f_{cu}} b_w d \qquad (18)$$

$$V_{Brown} = V_s + V_c = \frac{A_{sv}}{S_v} 0.95 f_{yv} d + \frac{1}{12}\sqrt{f_{cu}} b_w d \quad (19)$$

From Equation (18), V= V_{ACI} = 13.44 + 3.14 = 16.58 kN, and from Equation (19), V = V_{Brown} = 13.44 + 1.57 = 15.01 kN. From above the ACI and Brown models gave shear capacities which are about 136 % and 88 % greater than the average actual shear capacity of the space-framed models. However when compared with the result of Equation (17) which gives shear capacity due to concrete to be 3.99 kN, the ACI model predicts more accurately the shear capacity due to concrete V_C than that of the Brown model.

Table 2: Actual shear capacity for space-framed reinforced concrete with rigid beam-column joints

S/N	N_A(kN/m²)	$W = \frac{1}{4} N_A L_X$(kN/m)	$V_A = \frac{W}{2}$(kN)
1	55.20	13.8	6.9
2	53.36	13.34	6.67
3	57.04	14.26	7.13
4	58.88	14.72	7.36
5	60.72	15.18	7.59

5. CONCLUSION AND RECOMMENDATION

The cause of failure of the reinforced concrete space-framed models was by shear at the beam-column joints. The estimated shear capacity was 19.14 kN, while the average actual shear capacity was 7.13 kN, which is about 37% of the estimated shear capacity.

The above result shows that there is the need to carry out more research on reinforced concrete space-framed structures so as to obtain models that can predict more accurately the strength characteristics of reinforced concrete space-frame structures, especially the shear capacity at the beam-column joints.

6. REFERENCES

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