Shear Resistance of Steel Fiber Reinforced Concrete of Trapezoidal Shear Key

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Abstract. One of the concern that arise in precast concrete construction would be the connections of two components. Shear key is one of the method to connect two or more precast panels. However, there is no additional shear reinforcements were added at the shear key and failure is sudden due to the brittle characteristics of concrete. On the other hand, steel fibers reinforced concrete (SFRC) is known to have high resistance to cracking. The objective of this study is to determine the maximum horizontal load that can be applied to shear key connections when steel fibers were added and its mode of failure. The shear key used in this study is of the 45° trapezoidal shape. Meanwhile, the volume fraction of the steel fibers was applied at 0.5% and 1.0%. The specimens were tested using the "push-off" method. From the test results, the mode of failure, elastic and plastic behavior as well as stiffness analysis were obtained. All tested specimens experienced shearing failure. Although the results do not provide definitive conclusion due to the limitation of test parameters, the evidence indicated that there was no improvement in the shear strength with the addition of steel fibers. However, shear key with steel fibers was found to have better mode of failure where it did not failed prematurely or sudden as compared with the one without steel fibers. Furthermore, 0.5% addition of steel fibers has the highest stiffness value which shows its potential to provide strong resistant due to slip.

Introduction

Reinforced concrete is used in various construction because of its cost effective. In 1905, precast panel building was developed by John Alexander Brodie and Yannick Macken in order to save construction time at the same time lower the cost of building a house. Connections are the most vital part in prefabricated concrete constructions to ensure the rigidity and stability of a structure. These connections are needed to transmit high loading including both compressive and shear stresses at the joint. One of the method to connect two or more precast panels including the use of shear key. Shear key is use to increase the shear resistivity of the joint surfaces. Furthermore, the shape of shear key can affect its shear resistance capacity.

Plain concrete is known to have low tensile strength and strain. Fibers are normally added to overcome the application limit and at the same time increase the strength of concrete. Steel fiber reinforced concrete (SFRC) has high resistance to cracking and crack propagation, thus, the fibrous composites were developed so that it can provide improvement to the mechanical properties of the materials. Furthermore, steel fiber offers good tensile strength, impact strength and toughness [1].

Shear keys are usually unreinforced, and therefore the lack of reinforcement crossing the joint lead to low ductility and failure at joint area under high load. Therefore, this study was carried out to determine the maximum horizontal load that can be sustained by the shear key when different volume of steel fibers were added in the concrete and to investigate its mode of failure as well as the behaviour of shear key.

Previous Studies

Shape of Shear Key

Shear key is one of the method to connect two or more precast panels. Types of connection developed must provide adequate strength, ductility, and continuity to ensure the integrity of a structure [2]. Shear key connection is known to have a good shear resistance. When horizontal force is applied by the concrete segment, shear key provides strong resistant which was resulted from the combination of concrete, shear surface as well as its interlocking mechanisms [3].

The shape of shear key effect its shear resistance capacity. Key angles and shapes play a major role in producing high shear resistance. According to Hazman [4], the most effective shape of shear key connection is from the trapezoidal shape at a 45° angle. In comparison with the other 30° and 60° angles, the result shows that the trapezoidal shape with 45° has the highest elastic behavior and longest plasticity region.

Steel Fiber Reinforced Concrete (SFRC) Shear Key

Concrete is known for its brittleness and low tensile strength thus, attempts were made to control cracks by adding fibers in concrete. In 1847, Lambot came out with the idea of adding fibers into the concrete in the form or wires or wire mesh [5]. This led to the development of fibers as reinforcement to overcome the weakness of concrete.

Steel fibers are recognized for its ability to enhance the shear strength and prevent the development of cracks in concrete. Therefore it can control the brittle shear failure and at the same time improved ductility behaviour [6]. SFRC has good tensile and flexural strength, increased shock and fatigue resistance, ductility and also better crack arrest or crack control [7]. Previous investigation on SFRC shows that steel fiber improved ductility and provided better crack control in the shear-critical structure compared to that of using conventional shear reinforcements.

In addition to that, volume fraction of 0.5% to 1.0% of steel fiber content shows that it can effectively control the development of cracking similar to that of the conventional shear reinforcement is added [8]. Furthermore, another experimental study carried out by Vairagade, Kene, & Patil [9] found that value fraction of 0.5% and 1.5% were the most suitable proportions in the concrete mixture.

In this study, the purpose of adding steel fibers in the shear key without any region shear reinforcement was to evaluate its improvement in strength compared with conventional shear key. The improvement is not only to produce as high shear resistance but also to improve the failure mode. Steel fibers will limit the progression of cracking by transmitting the load across the cracked surfaces. Although adding steel fibers will not have high impact on the compressive strength, but they were known to provide better ductility [10].

Shear Strength Equations of Shear Key

Ultimate shear strength is the maximum shear stress that can be carried by an element [10]. According to Yang [11], AASHTO Recommendations [12] gives the following design equation for shear strength of shear keys.

$$Vj = A_{\mathrm{key}} \cdot \sqrt{6.792\ x\ 10^{-3} fck}\,(12 + 2.466 \cdot \sigma_{\mathrm{n}}) + 0.6 A_{\mathrm{sm}} \cdot \sigma_{\mathrm{n}} \tag{1}$$

where Asm is the area of contact between the smooth surfaces in the failure plane, Akey is the area of base in failure plane, σn is the average compressive stress across the joint (N/mm2) and fck is the concrete compressive strength (N/mm2). However, in German Recommendations, it mentioned that only frictional force is considered in the design. The equation is given as:

$$Vj = \mu \cdot \sigma_{\mathrm{n}} \cdot A_{\mathrm{T}} \tag{2}$$

where, AT is the effective shear area and friction coefficient, μ is taken as 0.7. Rombach [13] also proposed an equation to determine the shear capacity of dry joint given as:

$$Vj = (1/\gamma_{\mathrm{F}})(\mu \cdot \sigma_{\mathrm{n}} \cdot A_{\mathrm{joint}} + f \cdot f_{\mathrm{ck}} \cdot A_{\mathrm{key}}) \tag{3}$$

where friction coefficient, $\mu = 0.65$; safety coefficient, $\gamma F = 2.0$; Ajoint = area of compression zone; and factor for indentation of the joint, f = 0.14.

The shear capacity is Eq. (3) is the combination of both frictional and shear. The safety coefficient is taken as 2.0 as the failure characteristics of the joint is brittle. The load bearing capacity is influenced by the concrete tensile, compressive strength and the area of the failure plane, A_{key}.

Mode of Failure of shear Key

Based on the study by Buyukozturk [14], two types of cracks were identified known as S-crack and M-crack. S crack is the large single curvilinear while M-crack is the short or long diagonal multiple cracks. The S-crack and M-crack are shown in the Figure 1 below. High stress concentration will first developed at the upper edge of key. This is where the S-crack started to form and as the stress continued, the crack will continue to develop until it enters the low stress zone which is parallel to the original direction of the shear loading path. After that, rotation was initiated and leads to the development of stress field along the key. This resulted in tensile cracks referred as M-cracks. As the M-cracks continued, compressive stress between the M-cracks increases as well. Finally, the shear-off failure in the shear key took place resulted from the crushing of the compression struts.

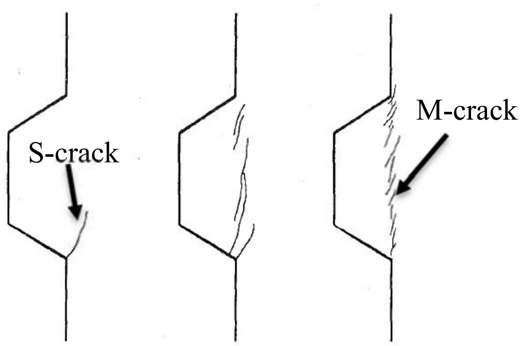


Figure 1: Cracking growth in shear key [14]

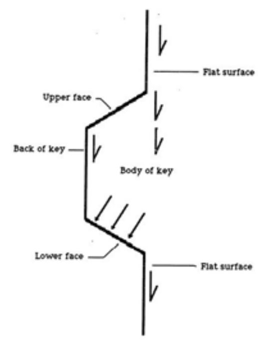


Figure 2: Uncracked concrete (Stage 1) [10]

Study on the behaviour of a single SFRC shear key was carried out by Beattie [10] using the “push-off” arrangement. According to Beattie, the study proved that by adding steel fibers in concrete, a more stronger and ductile concrete was formed. The shear transfer mechanism for Stage 1 is shown in Figure 2. The two principals for shear transfer are a friction and also the combination of both shear and compressive stress trajectories. Frictional forces formed along the flat surfaces. The addition of fiber does not really affect its frictional force where the results show that it produced similar pattern compared with plain concrete. However, the fiber addition affected the transmission of loading across the key and key body.

In Stage 2, cracking formed at about 30° angle to the shear plane as shown in Figure 3. The theory described that the compression is carried in the concrete through the struts that were formed between the diagonal cracks. For example, in the reinforced concrete using steel bar, the tensile force were carried by the reinforcement across the shearing plane. However, in keyed joint, the compression force was observed in the struts and while the tensile force existed elsewhere. Tensile components are said to be provided by the external prestressing. Results showed that the compressive strength in this stage was much lower than the standard cylinder strength. Although by adding fiber did not have much effect on the compressive strength, but they provided higher ductility.

In the final stage (Stage 3), cracked plane parallel to the loading direction were formed as shown in Figure 4. Along the flat surface, the frictional force was still formed but reduced due to the joint expansion. The loading was transferred through the key body at the lower key face. The unit strength is a function of fiber density, length, orientation and bond stress.

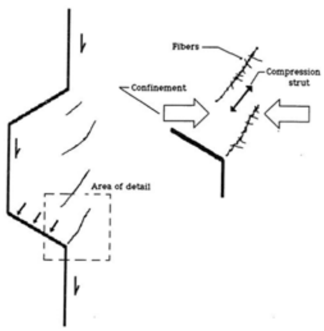


Figure 3: Diagonal cracking [10]

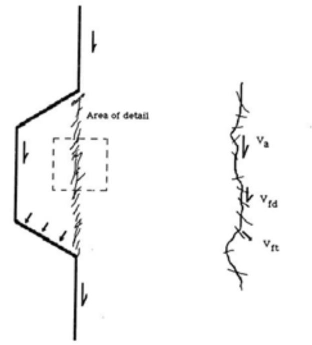


Figure 4: Final Cracking [10]

Experimental Work

This study focused on the influence of steel fiber in the shear key connection to determine the maximum horizontal applied load and its mode of failure. The results was then compared with the control shear key without any addition of steel fiber in the mixture (plain).

Details of Test Specimen and Concrete Properties

In this study, 3 sets of shear keys were prepared where each set consist of two panels. The bottom panel is the male part and the upper panel is the female part. The shear key was chosen from the trapezoidal shape which produced the highest shear strength based on the findings by previous researcher [4]. Figure 5 shows the dimension of specimen. The angle specified for the shear key was fixed at 45°. First set was the control (without adding any steel fibers) while the remaining two sets were added with volume fraction of 0.5% and 1.0% steel fibers. For every concrete batches of mixes, six cubes (150 mm x 150 mm x 150 mm) were prepared and tested at 28 days for compressive strength while three cylinders (150 mm diameter and 300 mm height) were prepared and tested for tensile strength at 28 days. The concrete mix was design in accordance to the Department of Environment (DOE) method. Table 1 shows the material proportions for 1 m^3 volume of concrete.

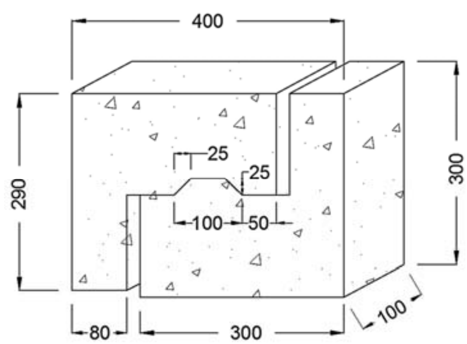


Figure 5: Specimen dimensions (in mm)

Table 1: Material proportion for 1 m3 volume of concrete

Concrete strength (N/mm^2)	Cement (kg)	w/c	Water (kg)	Fine agg. (kg)	Coarse agg. (kg)	Maximum aggregate size (mm)	Concrete slump (mm)
30	455	0.55	250	871.85	773.15	10	60 – 180

Casting of Concrete

Concrete strength used in this study was designed to achieve 30 N/mm2 at 28 days and the water-cement-ratio is 0.55. Meanwhile, the diameter and length of the steel fiber used is 0.75 mm and 60 mm respectively and the type of steel fiber used is hooked end. This gives an aspect ratio (L/d) of 80 which means that it is a long steel fiber. The procedure in mixing SFRC are as follows:

1. The drum surface was wetted with water before mixing.
2. Part of water was then poured in the mixer.
3. Cement, fine and coarse aggregates were added together in the mixer.
4. The balance of water was mixed with superplasticizers and added to the mixer.
5. Finally, steel fibers are slowly added into the mixer to avoid it from clumping together during mixing.

Test Setup and Testing Procedure

The strength of shear key was tested after 28 days using the push-off method as shown in Figure 6. The test was carried out using a loading frame. The slip movement at the shear key connection between the two panels can be measured using LVDT (Linear Voltage Displacement Transducer) as shown in Figure 7. The slip at the interface were calculated from the difference in movement between top and bottom panels. LVDTs were connected to data logger throughout the test. Roller is then placed on top of the specimen to avoid friction force. Load is applied horizontally as shown below to the shear key as shown in Figure 6 until it failed. Horizontal displacement was recorded at every 1.0 kN loading increment. The average loading rate applied to the specimens was 0.19 kN/s. The failure can be monitored when there is a sudden drop in load with a large increase in the horizontal slip. Failure can be defined when the shear key failed due to splitting and when multiple cracks were observed.

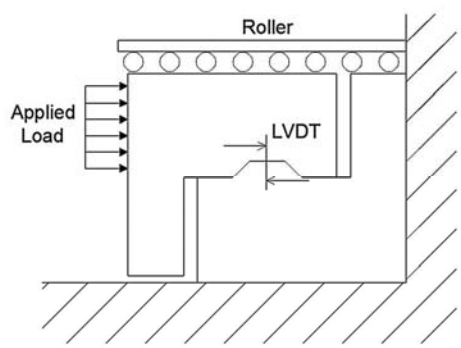


Figure 6: Schematic diagram of the "push-off" test setup

Figure 7: Position of LVDT

Results and Discussion

The test results include the concrete slump, compressive strength and tensile strength together with the shear strength from the "push-off" test. The test results are summarised in Table 2 and Table 3. The compressive strength at 28 days shows that all specimens reached the designed strength of 30 N/mm2. Compressive strength shows increment with the addition of 0.5% volume fraction of steel fibers but decreases at 1.0% volume fraction of steel fibers. The results shows that the tensile strength of concrete increases as the volume of steel fiber increases. The determination of this parameter is necessary to estimate the load at which the crack on the concrete may develop.

Table 2: Concrete Properties

Specimen	Volume of fiber	Compressive strength, N/mm^2	Tensile Strength, N/mm^2
A	0.0%	32.36	2.61
B	0.5%	46.67	3.53
C	1.0%	44.82	4.70

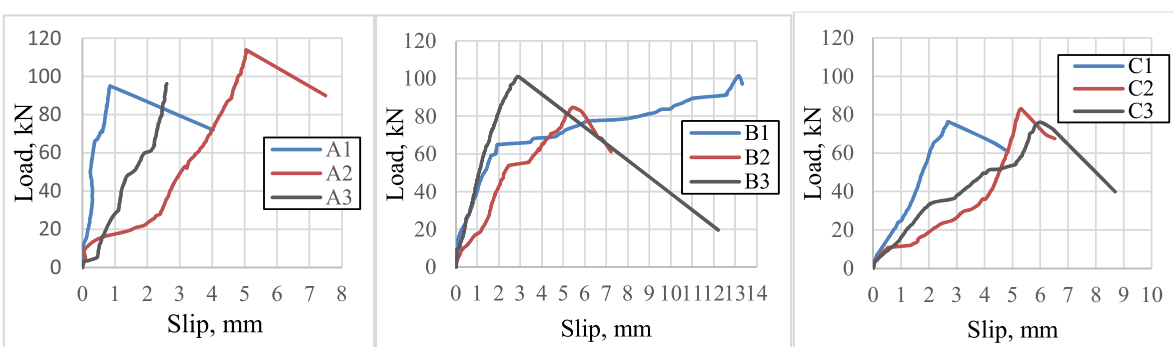
Maximum Shear Load Applied

Table 3 shows the maximum horizontal load at failure and the shear stress of the shear key. The relationship in Figure 8 shows that the maximum horizontal load decreases as the volume fraction of the steel fibers increases. The control specimen (0%) produced the highest horizontal load at failure. Shear stress at failure was obtained by dividing the horizontal failure load against the shearing failure area of the shear key.

$$\text{Shear stress, } \tau = \frac{\text{shear load, } V}{\text{Cross sectional Area of shearing failure}} \tag{4}$$

Table 3: Shear Stress at Failure of Shear Key

Specimen	Sample no.	Volume fraction of steel fiber	Horizontal load at failure, kN	Shear stress at failure, N/mm²	Average Shear stress at failure, N/mm²
	1		95.0	9.50	
A	2	0.0%	113.9	11.39	10.17
	3		96.2	9.62	
	1		101.5	10.15	
B	2	0.5%	84.7	8.47	9.57
	3		101.0	10.10	
	1		76.3	7.63	
C	2	1.0%	83.0	8.30	7.84
	3		76.0	7.60	



(a) (b) (c)

Figure 8: Applied horizontal load versus slip relationship of (a) specimen A, (b) specimen B, (c) specimen C

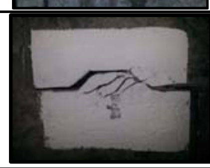
Specimen A which was the control produced the maximum horizontal load at failure of 95 kN specimen A1, 113.9 kN for specimen A2 and 96.2 kN for specimen A3. Meanwhile, the shear stress for specimen A1, A2, and A3 are 9.5 N/mm², 11.39 N/mm² and 9.62 N/mm², respectively. For Specimen B, the concrete mixture was added with 0.5% volume fraction of steel fibers. The results shows that specimen B1, B2 and B3 failed at 101.5 kN, 84.7 kN and 101 kN, respectively. The shear stress for specimen B1 was 10.15 N/mm² while shear stress for specimen B2 and B3 were

8.47 N/mm² and 10.1 N/mm², respectively. Lastly, Specimen C was added with 1.0% volume fraction of steel fibers. The maximum horizontal load for specimen C1, C2 and C3 were 76.3 kN, 83 kN and 76 kN, respectively. The maximum horizontal load for all three specimens had almost the same peak horizontal load, however they were found to be lower than specimen A and specimen B. Meanwhile, the shear stress for specimen C1 was 7.63 N/mm², specimen C2 was 8.3 N/mm² and specimen C3 was 7.6 N/mm².

Failure Mode of Shear Key

Table 4 shows the failure pattern of all specimens. Specimen A1, A2 and A3 show total splitting failure at the shear key connection. This is due to the reason that there is no shear reinforcement that can hold the two panels from total splitting. During the experiment, distinct crack line was formed at the shearing plane. The length of crack on the shear key increases until the shearing failure occurred followed by the splitting of the male segment. Sudden failure of splitting is not recommended because it could cause catastrophic failure when applied in the actual structure.

Table 4: Mode of Failure of Shear Key

Specimen	**Picture** 1	2	3	**Mode of Failure**
A				Shearing, splitting (all sample)
B				Shearing (all sample)
C				Shearing (all sample), crushing at key (sample C3)

However, the failure types for specimen B1, B2 and B3 are shearing failure. Specimen B2 experienced a little bit of crack at the edge of the top segment which caused the maximum shear load applied to be slightly lowered compared to specimen B1 and B3. Although the values are lower in comparison with the control specimen, the mode of failure of specimen B can be taken into consideration. When hacked into the key, the cracks only appeared on the outer surface as shown in Figure 9. The inside of keys was still intact due to the crack control mechanism of closely spaced steel fiber.

Figure 9: Specimen B after hacked

Figure 10: Crushing at key of Specimen C3

The failure mode of all Specimen C were similar to that of Specimen B. However, specimen C3 somehow failed due to crushing as shown in Figure 10. This is probably because of the steel fibers in the concrete were not distributed uniformly. Steel fibers were found to have higher resistance to cracking, thus this shows that steel fiber reinforced concrete has the ability of arresting crack growth.

The problem that may occur during the mixing of SFRC is the distribution of steel fiber in the mixture. It is important that the mixing with steel fibers was done carefully in order to produce a uniform distribution. Therefore, steel fibers were added lastly in the mixer after the other materials well mixed. Steel fibers tend to clump together when mixing in concrete due to the following reasons:

- Steel fibers already clumped together even before they were added in the mixture and therefore it cannot be break down by using normal mixing.
- Fibres was added too quickly during mixing.
- High volume of steel fibers were added.
- Steel fibers were added ahead in the mixer before the other materials.

Elastic and Plastic Behaviour of Shear Key

Ductile connector exhibit both elastic and plastic properties [15]. Elastic behaviour of shear key shows the tendency of shear key to return to its original position while yielding in the plastic region causes permanent deformation and subjected to the breaking of the shear key. Table 5 shows the data used for plasticity and elastic stiffness analysis.

Table 5: Data used for plasticity and elastic stiffness analysis

Specimen	Sample no.	Horizontal load at failure, kN	Shear stress at failure, N/mm²	Elasticity, kN/mm	Plasticity, kN/mm
A	A1	95.0	9.50	106.15	0
	A2	113.9	11.39	32.444	28.841
	A3	96.2	9.62	9.039	0
B	B1	101.5	10.15	31.175	2.7
	B2	84.7	8.47	20.681	13.643
	B3	101.0	10.10	36.722	0
C	C1	76.3	7.63	28.116	0
	C2	83.0	8.30	9.1188	40.458
	C3	76.0	7.60	16.646	11.948

Specimen A has no plastic region as the specimen failed instantly when it reached the maximum loading capacity, but specimen B and C display ductile behaviour. Both shear keys from specimen B and C were able to sustain the applied load before and after the specimen failed. Although the applied horizontal load reinforced with steel fibers is much lesser than the control, both Specimen B and C shows ductile pattern relationship. The control specimens which shows brittle failure probably due to higher energy stored in the system at high load.

Stiffness analysis was used to determine the shear key potential to resist horizontal movement. Stiffness of the shear key referred to the gradient tangent of the elasticity region. Specimen A had very large differences of elastic stiffness between specimen A1 (106.15 kN/mm), A2 (32.44 kN/mm) and A3 (9.04 kN/mm). The highest shear stress was from specimen A2 at 11.39 N/mm2. Meanwhile, specimen B1, B2 and B3 had consistent elastic stiffness at 31.18 kN/mm, 20.68 kN/mm and 36.72 kN/mm, respectively. Elastic stiffness for specimen B3 was found to be significantly higher compared with the other. This shows that specimen B3 had good interlocking strength resulted to strong resistant to horizontal slip. On the other hand, Specimen C had small differences in elastic stiffness which were 28.12 kN/mm for specimen C1, 9.12 kN/mm for C2 and 16.65 kN/mm for C3. Compared this with the other specimens including the control, specimen C3 has the lowest elastic stiffness.

Conclusion

The conclusions that can be drawn from the experimental test using the "push-off" method on the shear key performance with the addition of steel fibers are as follows:

1. The addition of steel fibers has less significant impact on the shear strength of the shear key. Specimen with 0.5% volume fraction of steel fibers failed at almost the same horizontal load with the control. However, the specimens with 1.0% volume fraction of steel fibers, plummeted to the lowest shear strength.
2. The 1.0% volume fraction of steel fibers was too congested resulted to the lower loading capacity of shear key. This is because high volume of steel fibers produced more stress concentration. Although steel fibers have the potential to provide higher ultimate strength, but it can decrease due to the formation of crack.
3. Although adding steel fibers to the shear key did not help to increase the ultimate shear strength, but the finding shows that it increased the ductile behaviour of the shear key. This can be seen from the results of Specimen B1 which have the longest length of plasticity region. Hence, it proved the increased ductility of the shear key.
4. Specimen B3 had better interlocking strength compared with the control specimen. This shows that the addition of steel fibers, provided strong resistant to the horizontal slip movement.

References

[1] ACI Committee 544, "State-of-the-Art Report on Fiber Reinforced Concrete," 2002.

[2] S. H. Rizkalla, R. L. Serrette, J. S. Heuvel and E. K. Attiogbe, "Multiple Shear Key Connections for Precast Shear Wall Panels," *PCI Journal,* pp. 104-120, 1989.

[3] X. Zhou, N. Mickleborough and Z. L, "Shear Strength of Joints in Precast Concrete Segmental Bridges," *Structural journal,* 2005.

[4] K. H. Padil, "The Ultimate Loading Capacity of Trapezoidal Shear Key in Precast Concrete Structure," Universiti Teknologi Malaysia, 2013.

[5] A. E. Naaman, "Fiber Reinforcement for Concrete," *Concrete International Journal,* pp. 21-25, 1985.

[6] American Concrete Institute (ACI) Committee 544, "State-of-the-art report of fiber reinforced concrete," *ACI 544,* 1996.

[7] V. S. Vairagade and K. S. Kene, "Introduction to Steel Fiber Reinforced Concrete on Engineering Performance of Concrete," *International Journal of Scientific & Technology Research Volume 1,* pp. 139-140, 2012.

[8] J. Susetyo, P. Gauvreau and F. J. Vecchio, "Effectiveness of Steel Fiber as Minimum Shear Reinforcement," *Structural Journal,* pp. 488-496, 2011.

[9] V. S. Vairagade, K. S. Kene and T. R. Patil, "A Study on Properties of Hybrid Fiber Reinforced Concrete," *Journal of Software & Hardware Research in Engineering,* pp. 1-3, 2012.

[10] S. M. Beattie, "Behavioral improvements in segmental concrete bridge joints through the use of steel fibers," Massachusetts Institute of Technology, 1989.

[11] In-Hwan Yang, Kyung-Cheol Kim, Young-Joon Kim, "Shear Strength of Dry Joints in Precast Concrete Modules," *Hokkaido University Collection of Scholarly and Academic Papers : HUSCAP,* 2013.

[12] AASHTO (1999). Guide Specifications for the Design and Construction of Segmental Concrete Bridges Second Edition , pp. 3-118.

[13] G. Rombach, "Precast segmental box girder bridges with external prestressing," *INSA Rennes: Segmental Bridges,* pp. 1-15, 2002.

[14] O. Buyukozturk, M. M. Bakhoum and S. M. Beattie, "Shear Behaviour of Joint in Precast," *Journal of Structural Engineering,* 1990.

[15] R. Hibbeler, Mechanics of Material Seventh Edition, Pearson Education South Asia Pte Ltd, 2008.
[16] L. W. P. M. R. A. K. M. S. J. a. W.A.Thanoon, "The Experiences of Malaysia and Other Countries in Industrialized Building System in Malaysia," in *International Conference on Industrialised Building Systems*, Kuala Lumpur, 2003.
[17] J. Turmoa, G. Ramosb and A. Aparicio, "Shear strength of dry joints of concrete panels with and without steel fibres," *Engineering Structures,* 2005.