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SHEAR BEHAVIOUR OF ULTRA HIGH PERFORMANCE CONCRETE UHPC TAPERED BEAMS WITH LONGITUDINAL OPENINGS

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بِسۡمِ ٱللَّهِ ٱلرَّحۡمَٰنِ ٱلرَّحِيمِ (1)
ٱلۡحَمۡدُ لِلَّهِ رَبِّ ٱلۡعَٰلَمِينَ (2)
ٱلرَّحۡمَٰنِ ٱلرَّحِيمِ (3) مَٰلِكِ يَوۡمِ ٱلدِّينِ (4)
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ٱلصِّرَٰطَ ٱلۡمُسۡتَقِيمَ (6) صِرَٰطَ ٱلَّذِينَ أَنۡعَمۡتَ
عَلَيۡهِمۡ غَيۡرِ ٱلۡمَغۡضُوبِ عَلَيۡهِمۡ
وَلَا ٱلضَّآلِّينَ (7)

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To my supervisor Dr. Nasser Hakeem

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ABSTRACT

The work in this thesis is divided into two parts, the first about ultra high performance concrete UHPC, it's concrete's type that was advanced could enhance the concrete structures' resilience and durability. The local materials utilizing are essential stride to materials saving, energy, and concrete's cost reducing. In current study, binder's content, w/c ratio, and sand's gradations on concrete's compressive strength were examined. It's conceivable to promote UHPC mixtures from available locally materials by utilized three types of sand (sand #2, sand #3, and sand #4), were indicated cube strength of (163.2MPa, 164.8MPa, and 167MPa) were achieved with total binder content (1250kg/m3, 1300kg/m3, and 1300 kg/m3) from (sand#3) with (25%, 30%, and 30%) respectively of silica fume. The cylindrical strength of (160MPa, 150.9 MPa, and 158.8 MPa) with total binder content (1337.7 kg/m3, 1250 kg/m3, and 1337.7 kg/m3) from three types of sand (sand #2, sand #3, and sand #4) respectively, with (30%, 25%, and 30%) respectively of silica fume. As obtain the cylindrical specimens compressive strength were about 12% lesser than cube specimens' compressive strength. After getting the UHPC mixture then utilized in second part (main aim) that casting nineteen tapered-beams (twelfth groups, were tested under two point loads) with the following parameters NS CFRP bars with three orientations (0˚, 30˚, and 45˚), CFRP strips U-wrapped with same NS orientations, stirrups number, tapered-beams inclination, shear span to effective depth a/d ratio, tensile bars ratio, steel fiber ratio, and openings number and position. Results showed that the inclined CFRP bars/strips were more efficiency than vertical ones. NS was most effectiveness, not only increasing tapered-beam's shear strength by (11.3%, 35.4 %, 36.6 %) with orientations (0˚, 30°, 45°) respectively, but also increasing first cracking load (3.3%, 43.7%, and 57.4%) respectively, service deflection increases by (210%, 222%, and 225%) respectively, and deflection increased by (11.5%, 85.6%, and 99%) respectively. The NS is more effective than stirrups in increasing shear strength, first cracking load, and deflection by (13.9%, 18%, and 53.4%) respectively, comparing with same number of rods, instead of the stirrups diameter was 8mm and NS CFRP deformed bar was 6mm. NS CFRP bar is more aptitude than CFRP strip in all orientations in increasing of ultimate load capacity, first cracking load, service deflection, and final deflection by (21.1%, 40.5%, 73.3%, and 93.8%) respectively for 45˚ orientation. Tapered-beam's shear strength is effected by stirrups number and it increased from 19.9% to 30.8% when utilized 5 stirrups instead of 4 stirrups, and the first cracking load, service deflection, and deflection increased by (43.7%, 240%, and 56.5%) respectively, when stirrups number increased from (0 to 5). Inclination had positive affect on tapered-beams shear

strength. The failure load, first crack load, and deflection, were increased by (19.3%, 24.5%, and 86.5%) respectively when inclination angle increased from (9.7° to 15.9°). The increasing a/d led to decrease tapered-beam shear strength. When a/d decreased from 2.94 to 2.3 led to increase of failure loads and deflection about 10.6% and 50.4% respectively. Tensile bars had positive affect especially when bars distributed by two rows due to the dowel action affects, when steel's area varied from (981.7mm^2 in one row) to (804.2mm^2 in two rows) shear strength increased by 3.2% despite of steel area was lesser by 18 %, but distributed by two rows, also the first cracking load, service deflection, and deflection increased by (12%, 90.2%, and 6.6%) respectively. When tensile bars ratio increased from 1.22% to 1.57% when tensile bars distributed by two rows, the ultimate load, first crack load, service deflection, and deflection increased by (20.5%, 26.5%, 27%, and 41%) respectively. Steel fiber 2% had excellent effective on increasing tapered-beams shear strength by 300 %, which means better than stirrups even than CFRP bar/strip. Steel fiber presences led to significant increasing in load value at which first crack appeared 84.7%, and increased deflection by 235%. The opening had negative affect on tapered-beams shear strength not just in load carrying capacity, but also on first crack load, deflection, and service deflection by (5.2%, 18.2%, 13.5%, and 19%) respectively. The tapered-beam with one opening in prismatic region has the same shear capacity and deflection of the tapered-beam with two openings. The failure angle ranges from 31.197° to 36.297° for tapered-beams without (stirrups and CFRP bars/strips). As for tapered-beams those were with (CFRP bars/strips and stirrups), the ranges of failure angle were 41.197° to 52.797°. Finally, the designing was done by three methods (deep beam, irregular section, and Nasser's formulas with Albegmprli et al. formula to calculate dowel action contribution in shear capacity, and compared with experimental results for all nineteen tapered-beams was Nasser's formulas is suitable for designing this type of beam with mean conformity ratio 93.3%.

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NOMENCLATURE

The major symbols used in the text are listed below; others are indicated with their equations where they first appear.

ABBREVIATIONS	
AASHTO	American Association of State Highway and Transportation Officials
ACI	American Concrete Institute
AFGC	Association Française de Génie Civil
AFRP	Aramid Fiber Reinforced Polymer
ASTM	American Society for Testing and Materials
CFRP	Carbon Fiber Reinforced Polymer
DIN	Deutsches Institut für Normung
EBR	External Bonded Reinforced
FHWA	Federal Highway Administration
FRP	Fiber Reinforced Polymer
GFRP	Glass Fiber Reinforced Polymer
HRWRA	High-Range Water Reducers Admixture
HSC	High Strength Concrete
HT	Heat Treatment
ISIS	Intelligent Sensing for Innovative Structures
ITZ	Interfacial Transition Zone
JSCE	Japan Society of Civil Engineers
MDF	Macro Defect Free
NS	Near Surface
NSM	Near Surface Mount
PVC	Poly Vinyl Chloride
RC	Reinforced Concrete
RCHB	Reinforced Concrete Haunched Beam
RO	Reverse Osmosis
RPC	Reactive Powder Concrete
SF	Silica Fume
UHPC	Ultra High Performance Concrete
UHPFRC	Steel Fiber Ultra High Performance Concrete
ULS	Ultimate Limit State

SYMBOLS	
A_{fv}	Area of FRP shear reinforcement within spacing (s) (mm^2)
A_s	Area of tension steel
a/d	Shear span to effective depth
b	Width of rectangular cross section
B_u	Diagonal crack inclination ≥ 30°
C_E	Environmental reduction factor
d	Distance from extreme compression fiber to centroid of tension reinforcement
d_b	Diameter of reinforcing bar
d_{fv}	Effective depth of FRP shear reinforcement
d_{net}	Reduced length of (FRP rods)
d_r	height of shear strengthened part of cross section
E_c	Modulus of elasticity of concrete
E_f	Tensile modulus of elasticity of FRP
f_c'	Specified compressive strength of concrete
f_{fe}	Effective stress in the FRP; stress level attained at section failure
F_{vd}	UHPC tensile strength
h	Thickness of the cross section
k_1	Modification factor applied to κ_v to account for concrete strength
k_2	Modification factor applied to κ_v to account for wrapping scheme
κ_v	Bond dependent coefficient for shear
L_e	Active bond length of FRP laminate
L_i	Rod's Effective length crossed by crack corresponding to tensile strain
L_{tot}	Sum of effective lengths of all rods those crossed by crack
n_{tf}	Applied number of FRP sheet/strip layers
s_f	Span between each sheet
t_{frp}	Nominal thickness of one ply of FRP reinforcement
V^a_{Rd}	Design value of shear bearing capacity of Tapered-beams at design section
V_{ccd}	Design shear resistance due to inclination of compression chord of beam
V_c	Nominal shear strength provided by concrete with steel flexural reinforcement
V_{Ed0}	Shear force due to dead loads and live loads
V_{da}	Dowel action of inclined main reinforcement
V_{1F}	FRP shear strength contribution concerning to bonding shear failure

V_{2F}	*FRP shear strength contribution corresponding to maximum FRP strain*
V_{fd}	*Designed shear capacity that provided by steel fibers*
V_{frp}	*The contribution of external FRP to shear capacity*
V_s	*Nominal shear strength provided by steel stirrups*
V_{td}	*Design shear resistance component of inclined longitudinal tension reinforcements*
V_{pd}	*Design shear resistance component of prestressed force*
V_{ped}	*Component of the tendons effective tensile force (parallel to the shear force)*
w/b	*Water to binder ratio*
w/c	*Water to cement ratio*
W_f	*Width of FRP reinforcing plies*
Z	*Lever arm*
α_1	*Ratio of average stress of equivalent rectangular stress block to* f_c'
γ_b	*Safety factor = 1.3*
γ_E	*Safety coefficient*
β	*Ratio of distance from neutral axis to extreme tension fiber to distance from neutral axis to center of tensile reinforcement*
β_1	*Factor relating depth of equivalent rectangular compressive stress block to neutral axis depth*
ρ_{frp}	*FRP reinforcement ratio*
ρ_l	*The ratio of longitudinal tensile reinforcement*
τ_b	*Average bond strength*
σ_m	*Mean stress in the concrete total section under normal design force*
σp	*Residual tensile strength*
ε_{fe}	*Effective strain level in FRP reinforcement attained at failure*
ε_{fu}	*Design rupture strain of FRP reinforcement*
ψ_f	*FRP strength reduction factor*

CHAPTER ONE

INTRODUCTION

1-1 GENERAL

Tapered-beams are widely utilized in the continuous and simply supported bridges, portal frames, cantilevers and buildings with midrise framed. Such beams could reduce structures weight and contribution in the appearance from aesthetic view point, while utilizing the steel bars and concrete more efficiently. Occasionally, they are likewise utilized to facility the services placement and equipment (piping, air conditioning, electrical, etc.) by as long as extra space under ceiling (economical building construction). They are an attractive solution of structural with large spans or bay widths, in the same time it needs to form over work and specialist workforces needed. (Colunga et al. 2017) [1].

The cross-section of any beam could be made tapered by varying depth, width, or both continuously or dis-continuously over its length. Width variation causes more construction difficulty. Thus, beam with depth varying is generally provided. Effective depth of such beam is varying from point to point. (Jolly et al. 2016) [2].

Tapered-beam is utilized as (cantilever, continuous, and simply supported beam) in buildings, and bridges (Fig. 1-1) [1]. Member that doesn't have the same cross-sectional properties from one end to the other, that doesn't have a straight axis, and that has reinforcement over parts of their lengths are known as non-prismatic beams. Despite of tapered-beams are commonly utilized in mid-rise building and bridges, there aren't specific recommendation for design such beams in (ACI-318-14) [3] & (BSI 1988) [4]. So, tapered-beam's design has been left to the structural engineers' experience and judgment in professional practice (Arturo et al. 2008) [5].

Fig. 1-1 Concrete tapered-beam [1].

The openings existence in the beams are more often utilized to provide passage for utility pipes and ducts are utilized in electricity, air conditioning, water supply, gas lines, and sewage, in the same time its translate into substantial of economical saving in the multi-stories building construction, and also aesthetic reasons. The ducts must have covered with suspended ceiling, if the beam hasn't opening this leads to increase the floor high, in the small buildings that increase may doesn't be significant in compared with overall cost, but in the multistory buildings the small saving in the height of story multiplied by the stories number which leads to paramount saving in the total height, and the construction cost will be decreasing. (Elnady 2015) [6].

In the design of concrete members, the area of steel reinforcement (in the tensile zone) is always calculated on the basis that the steel reaches the yield by utilizing steel area less than that causes the balance failure, by limits the used maximum percentage of the steel does not exceed 0.75 ρ_b, or by restrict the steel strain value not less than 0.004 (Jamal Alesawi 2005) [7], and these two determinants are to ensure that failure occurs firstly in steel for the purpose of the occurrence of elastic failure, and also to ensure the happen of failure indicators, such as deflection and cracking.

As for Shear is considered one of the major factors that affecting the structural elements stability. Structural elements those subjected to shear bear diagonal tension and brittle failure which maybe result to suddenly collapse of the building (Ahmad et al. 1995) [8].

1-2 Behaviour of concrete's members under shear

Concrete's elements those under shear load at ultimate capacity constantly have shear cracks. Shear cracks could have generated in beams' web the region of maximum shear stresses. Shear cracks progressing from previous flexural cracks denominate flexure shear cracks (Fig. 1-2 d) (Pillai et al. 2005) [9].

The failure's type brings about by those cracks, ordinarily in highly brittle, with suddenly way, denominate shear failures. Normally, there're five various styles failure bring about diagonal cracks count on the geometries, dimensions, loading kind, tensile bars amount, and concrete members' structural characteristics (Fig. 1-2) as following: (1) Shear compression failure, (2) Diagonal tension failure, (3) Shear tension failure (4) Web crushing-failure, and (5) Arch rib failure.

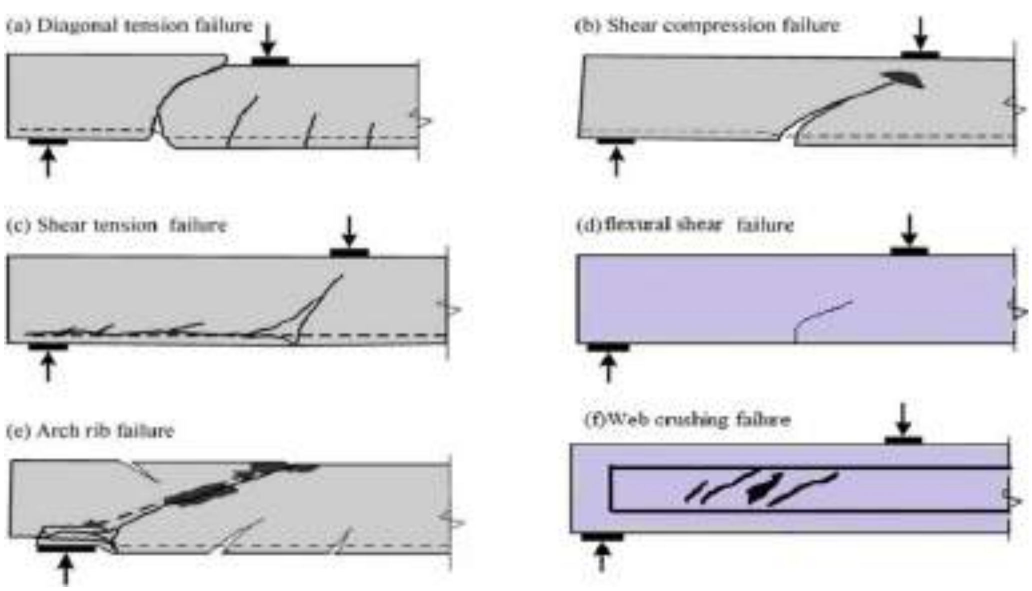


Fig. 1-2 Shear failure modes of concrete beams [9].

Diagonal tension failures ordinarily occur in the concrete’s elements had low quantity of transverse reinforcement, and tensile' bars. Diagonal cracks might have generated from preceding flexural crack and propagate quickly over all element's section till collapse (Fig. 1-2.a). For concrete’s elements had low quantity of transverse reinforcement but sufficient tensile bars to compressions zone's forming, shear cracks might readily generate from preceding flexural crack, but don't passing over compressions zones. Structure's failures are bringing about by concrete's crushing in compressions zones over shear crack's tip and denominate shear compression failure (Fig. 1-2.b). In statuses that lose bonding with concrete because inappropriate tensile' bars' anchorage or concrete's cover, crack tends to appears along tensile' bars till cracks merge with flexural shear cracks to bring about shear tension failures (Fig. 1-2.c). In I-beams only specified Crushing web's failures appears; because slim web's thickness, whilst failures of arch action ordinarily occur in the deep beam (Fig. 1-2.f and 1-2.e).

1-2-1 Mechanisms of shear transfer

How shear could have transferred, and which concrete’s structure parts under shear are still appear to be challenging to community of researches because intricate physical mechanisms those don't follows to no classical theory's mechanical. Although some shear resistance basal action in the concrete's structure are known inclusive: (1) shear resistance of un-cracked compressions concrete's zones (Shear stress that occur in compressions zones); (2) aggregate interlock; (3) longitudinal reinforcement dowel action; (4) Residual of tensile stress transmit immediately across the crack (crack bridging tension forces existent in the closed cracks); and (5) arch action (ACI 445R-99) [10] as illustrated in (Fig. 1-3), nevertheless, the importance level of each conformable shear resistance action yet debating.

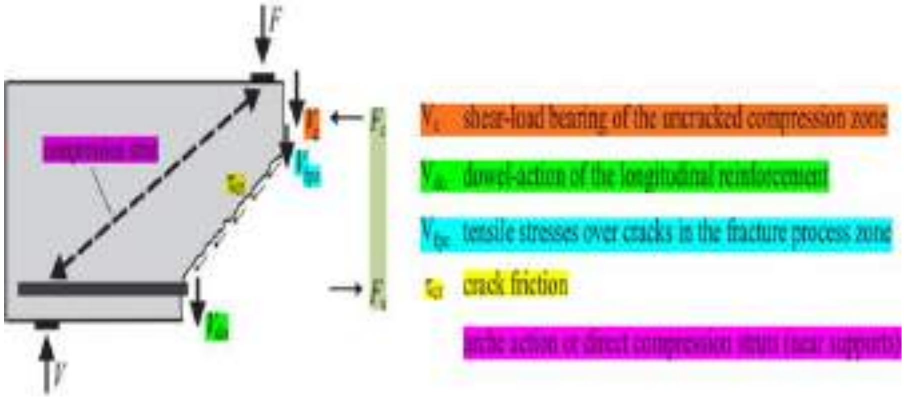


Fig. 1-3 Mechanisms of shear load transfer.

1-2-1-1 Shear resistance of un-cracked compression concrete zone

As shown in (Fig. 1-2.a, and 1-2.b) (Pillai et al. 2005) [9], concrete's compressions zones plays substantial function in the limiting and guidance of inclined cracks evolvement. It's obvious that concrete members fail in shear due to cracks of shear. Failure is occurring when critical shear cracks surpass concrete compressive strengths or pass through compressions zones. Therefore, the compression zone thickness will determine the member's load bearing capacity. The greater compression zone's thickness, the greater concrete shear capacity.

1-2-1-2 Friction of contact surfaces between cracks

Cracks surfaces' coarseness acting as interlocks to deny slid among the contacting surfaces. The mechanism denominates (aggregates interlock) depending on cracks widths and size of aggregate. Shear resistances enhances when utilize greater size of aggregates and cracks widths' decrease, shear forces are fundamentally transfer by two mechanisms (interlocks and dowel action) (Reineck 1991) [11]. In disparity, (Zararis et al. 2001) [12] declared that due to un-cracked concrete's regions exist on top of critical inclined crack tips, its act as buffer to deny each slid along interfaces crack. So, there aren't participation of dowel action and aggregate interlock.

1-2-1-3 Dowel action

Recent works by (Tassios and Vintzeleou 1987) [13], and Chana (1987) [14], had reaffirm the renowned action by (Baumann & Rüsch, 1970) [15], about dowels' resistance nigh surface. Normally, dowel action isn't very considerable in element hasn't stirrups, because the dowel's ultimate shear is restricted by concrete's cover tensile strengths that supports dowels. Dowel action possibly considerable in elements with huge tensile bars amount, essentially when tensile' bars distributed with more one row. When develops of critical shear cracks and progressively wider of prior flexural crack, major tensile' bar will resist shear by acts as a dowel. Shear resistances counts on critical shear crack's columnar displacement and mostly on concrete's beam thickness and concrete's tensile strengths. (Watstein et al. 1958) [16], were inference test nine beams with no stirrups, then deduced [tensile' bars carried about (0.38 to 0.74) from overall shear when load ranges (0.42 to 0.46) of ultimate. Then dowel action decreases when shear crack became wider to be zero at fail. (Acharya et al. 1965) [17] are tested (20 beams) without transversal bars, their inference was (dowel action not just shear's carrying, but even functions a major role in governing which type of failure shear or flexure failures, would happening).

1-2-1-4 Residual of tensile stress transmit immediately across the crack

Residual of tensile stresses' basic explanation is that when concrete first cracks, clean break doesn't occur. Small concrete’s pieces bridge the cracks and continue to transmit tensile forces up to crack widths in range of [0.05-0.15mm]. Experiential investigates by (Gopalaratnam et al. 1985) [18], inference that cracked concrete is resisting tensile. Subsequently, concrete's teeny portions crossing the crack able to carrying shear as long as cracks widths doesn't surpass appoint limited values, the greater the crack's width, the minimal considerable the shear capacity of concrete's cracked portions. However, (Bazant, 1997) [19], in theory verified that (the tensile stress of cracks bridges are trivial, so must ignored because its quantum is highly lesser than concrete's compressions zones' shear capacity).

1-2-1-5 Arch action

The concrete elements' shear resistances could divide for two separated styles: beam action, where the forces in tensile bars are acts on lever arm's modification to create an equilibrium with external moment, and the arch actions, where lever arm (locus of longitudinal compression stress generating in concrete) modification to balance the moment. The choice's method for statuses predominate by arch actions is struts models and ties models. Concerning beams action, the physical model and mechanical model could have classified further into tooth models, which begin with assumed cracks arrangement, and truss model with smeared concrete tensile field or concrete tie [10]. (Fig. 1-4).

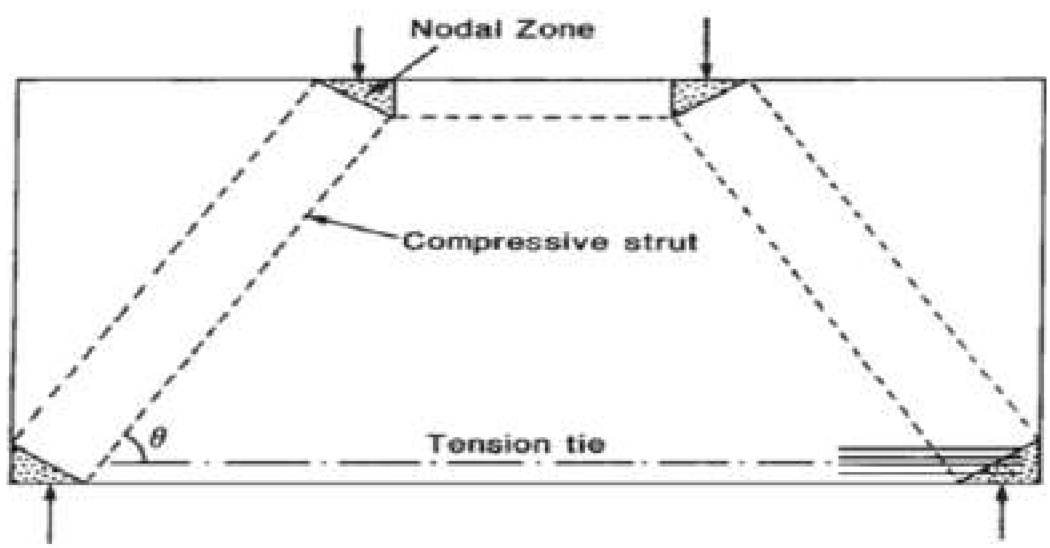


Fig. 1-4 Details of arch action.

1-3 Fiber Reinforced Polymer (FRP)

FRP is a composite material consist of fibers with polymeric resin. Resin acts like a binder material, holds polymer fibers in intentional position, protection, and giving the structural integrity.

At present time, fiber reinforced polymer FRP utilized as external bonded in the members repairing to strengthen and retrofitting existing structures due to its high strength to weight ratio, excellent durability, non-magnetic, non-corrosive, easiness in application, extra to chemicals resistant, so FRP represents perfect choice in external reinforcement. (Kiang 2003) [20].

1-4 FRP types

1-4-1 Glass Fiber Reinforced Polymer (GFRP)

Its properties are same of glass material properties, and it has the lowest price of other types. The E-glass [(E) means electrical] for electrical work utilizing, it's one of the most commonly utilized glass fibers because it's the most economical, C-glass means higher corrosion resistance, AR-glass means alkali resistance. (Sathishkumar 2014) [21].

1-4-2 Aramid Fiber Reinforced Polymer (AFRP)

This type of fibers has high energy assimilation during failure, and this property makes AFRP suitable to utilize versus impact and bulletproof vest, ballistic protection and aircraft. AFRP provides high tensile strength and extraordinary flexibility. Its best choice as structural material for resisting vibration and high stresses. (Denchev et al. 2012) [22].

1-4-3 Carbon Fiber Reinforced Polymer (CFRP)

CFRP defines as a fibers containing at least 90% carbon by weight. It consisting of carbon atoms, and fine fibers, each has size of diameter (0.005mm –0.01mm), and those carbon atoms connected together by microscopical crystals along the direction of the fibers. By another words CFRP is a composite materials which rely on carbon fibers to provide strength and stiffness while the polymer provides cohesive matrix to protect and hold fibers together. This type of polymer fibers has high strength to weight ratio, modulus of elasticity to weight ratio, easy to construction and handling, rapid project delivery. Thus, the fibers are strong enough, its ultimate tensile strength is very high to weight ratio, and its modulus of elasticity is thrice of steel's ones (Sharun et al. 2019) [23], and it's available as bars, and laminates. Some of disadvantages of CFRP include high cost, electrical conductivity, and high brittleness, which might limit CFRP application potential.

(Fig. 1-5) shows the FRP types (Nasser 2016) [24].

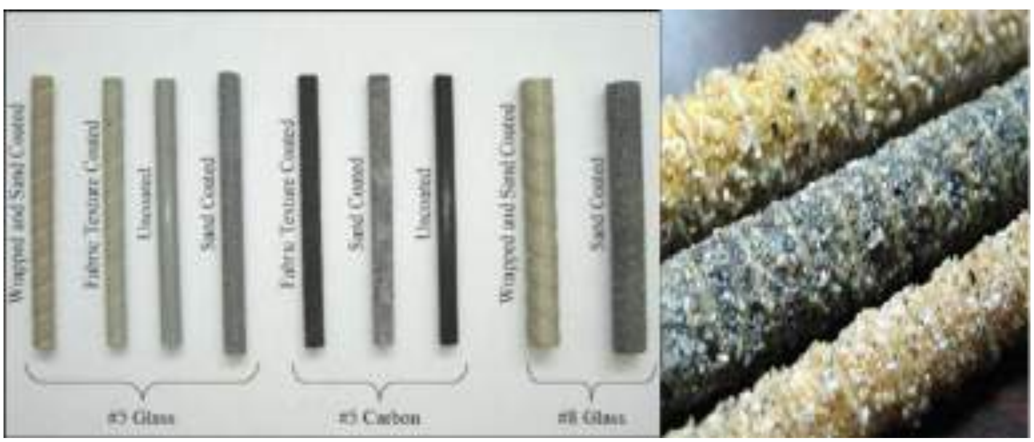


Fig. 1-5 FRP types [24].

1-5 Applications of (CFRP)

The CFRP implementation fields include buildings, tunnels, bridges, and others such as box culverts, electric poles, and among others. Out of what mentioned, implementations in buildings and bridges occupy majority of whole market. Newly, more implementations can be found in the repairing of tunnel lining. FRP utilize to strengthen of existing structures or as an alternative reinforcing or pre-stressing material instead of steel reinforcement from the onset of project. Bridges retrofitting application have witnessed the great success owing to utilize of CFRP composite technology. Retrofitting technique is widespread in numerous instances, the replacing cost of deficient structure could be extremely exceeding its strengthening utilizing CFRP, old concrete bridge decks those had been reinforced with unprotected steel reinforcement are deteriorating rapidly, the CFRP composite deck system has the possibility to fill the need of bridge deck replacement and extend the service life of existing structures (Elisa et al. 2001) [25]. Wrapping around bridge sections can also enhancing the section ductility, greatly increasing of collapse resistance under earthquake loading. Seismic retrofit is the major application in the earthquake prone areas, since it's much more economic than alternative methods. The wrap of fiber system is also being utilized to repair a deteriorated concrete pier, decks, retaining wall, pier caps, damaged beams, piles, and concrete arch (Milliken 2017) [26]. Bonded concrete repair utilizing CFRP rods, laminates and wet layup fabrics are also very widespread repair technique. Two techniques could be adopted to beams strengthen. First is to paste CFRP plates to beam's bottom generally in tension face, this is increase the beam's strength, beam's deflection capacity and stiffness. Alternatively, CFRP bar/strip could be pasted in U-shape around the beam's bottom and sides, resulting in higher shear resistance. Building's columns could be wrapped with CFRP for achieving higher strength. This technique is work by restraining the lateral expansion of column. Slabs might be strengthened by pasting CFRP strips at their bottom tension face, better performance will result, since the tensile resistance of slab supplemented by tensile strength of CFRP, effectiveness of CFRP strengthening is depending on the resin performance that chosen for bonding. (Hota et al. 2007) [27].

1-6 Near Surface (NS) CFRP technique

In the last years, strengthening technique depended on NS of laminate strips/bars of CFRP has been utilized to increase concrete members' load bearing capacity, the expression "near" is utilized to distinguish this technique of structural strengthening from that utilizing externally bonded (EBR). In NS CFRP technique, laminate strips/bars of CFRP are inserted into potholes pre-cut on elements' concrete cover to be strengthened those were formerly full of with epoxy adhesive (Cruz et al. 2004) [28], (Fig. 1-6) shows applications of NS CFRP bars, and CFRP strip/ bars techniques.

(a)

(b)

(c)

Fig. 1-6 FRP Application (a, b, c, e) near surface bars, (d) strips [28].

(d) (e)

Continue Fig. 1-6 FRP Application (a, b, c, e) near surface bars (d) strips [28].

1-7 Ultra High Performance Concrete UHPC

Concrete technology has made salient advances in the recent decades. Over the past (20 - years), several of working in this field have developed UHPC up to the level where they're ready to implementation. And with compressive strengths (150 MPa - 200MPa) (Association Francsaise de Genie Civil AFGC 2002) [29], (120 MPa – 400MPa) (Canadian Highway Bridge Design 2007) [30], started from 120 MPa (Schmidt et al. 2004) [31]. It's reinforced with the fine steel fibers with high tensile strength, such concrete becomes ductile, and reach tensile strengths up to 50MPa. So UHPC can for first time to be designed to the accommodate tension, and by utilizing new design essentials suited to UHPC, with or without traditional reinforcement, the resulting is forms of the concrete construction that saves materials and are so especially sustainable, it isn't only UHPC strength that's high, compared with the conventional normal concrete and HSC with their capillary porosity, UHPC offers much denser micro-structure. Virtually it hasn't capillary pores, thus UHPC impervious to the liquids, and gases so its corrosion practically zero, UHPC can serve as wearing course of bridge deck without other additional protection against alkalis, chlorides or salts deicing. Owing to UHPC good durability, materials saving composition, and low maintenance requirement, structure made from UHPC when properly designed [29]. The basic idea of concrete producing with very high strength, and especially dense micro-structure had formerly been put forward in (1980s). But practically break through was came with development of an especially efficient superplasticizer that enabled concrete production with high proportion of optimally tightly packed ultrafine particles and extremely low water / low binder ratio about (20%) in easy flowing consistency, the optimum combination of those two principles is what gives UHPC it's special characteristics. To produce UHPC with compressive strength (150MPa –200MPa), it's important to observance following basic rules:

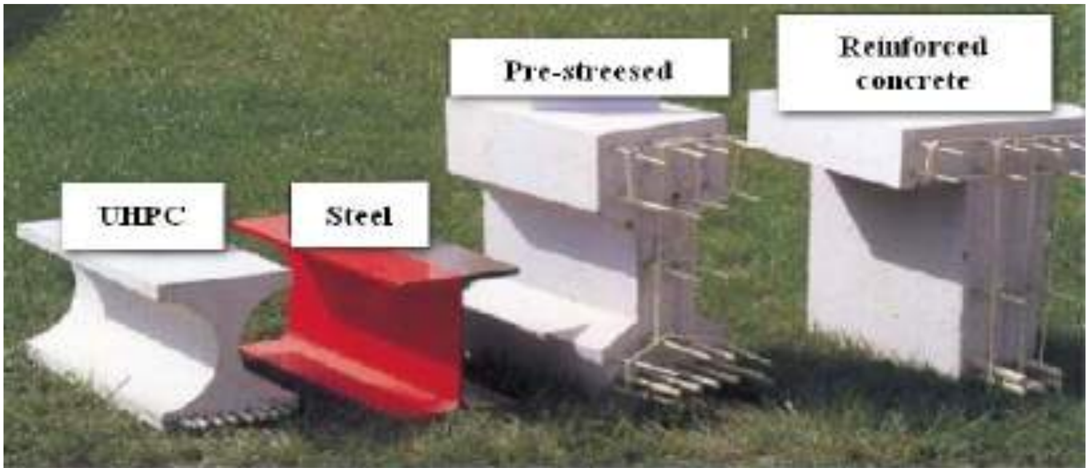
- Maximum grain size must be less than that of traditional concrete mix because large-grains cause concentrations of stress that leads to strength decreasing. These days, maximum grain size for UHPC usually not larger than (2mm). However, UHPC with maximum grain size of (8mm) has also been developed.

- Optimum packing density of aggregates are very important. High packing density could be obtained with help of fine materials, which reducing stress on contact surface and ensure that micro-crack doesn't begin to form till higher level of stresses are reached. The micro-structure is very dense which expresses itself not only in high strength, but also in much higher resistance to whole forms of attacks those damage reinforcements or concrete (alkalis, chloride, de-icing salts, and carbonation).

- The cement amount that utilized must be such that a water is fully bound. The remaining non-hydrated cement particles then act as filler.

- Steel fibers must be utilized to guaranty the ductility.

This new concrete material is consisting of cement, sand with maximum size of (600μm) sieve opening, high tensile steel fibers with appropriate aspect ratio, silica fume, superplasticizer, and low water to cement or low water to binder ratios are utilized to producing this concrete kind (Fehling et al. 2013) [32]. (Fig. 1-7) shows the difference between UHPC and other materials by weight and depth.



Material	UHPC	Steel	Prestressed Concrete	Reinforced Concrete
Depth	14 in. (360 mm)	14 in. (360 mm)	28 in. (700 mm)	28 in. (700 mm)
Weight	94 lb/ft (141 kg/m)	75 lb/ft (110 kg/m)	313 lb/ft (466 kg/m)	355 lb/ft (528 kg/m)

Fig. 1-7 Comparison among UHPC and other materials by weight and depth [24].

It seems that shortage of designing codes, restricted knowledge on together technologies (production and material), with high costs are restrict the application of this distinguished material beyond the initial demonstration projects.

Cement, cementitious materials (fly ash, silica fume, and cement slag), glass or quartz powder, fine sand, steel fibers, low water content, and High Range Water Reducing HRWR are UHPC's mixture proportions (Graybeal 2007) [33].

In numerous UHPC mixture's proportions coarse aggregate was excluded, this exemption minimizes micro-cracks those existing in coarse aggregate, also in interfacial transition zone ITZ between coarse aggregate, and matrix's paste. Those micro-cracks could enhance concrete permeability (Cornelia et al. 2012) [34].

Moreover, when the concrete resisting applied loads, mechanical cracks tend to occurs at situated micro-crack and propagated through coarse aggregate and matrix's paste which could produce concrete's failure. Subsequently, coarse aggregate's exemption is substantial to ameliorate UHPC’s durability and strength.

Coarse aggregate elimination combined to granular mixture' optimization permits obtaining intense and homogeneous cementitious matrix that offers high mechanical performance (Richard et al. 1995) [35].

Premix of UHPC obtainable in numerous international markets (Graybeal 2006) [36], Premix needs special concern throughout mixing process, pouring, treatment and testing. Like, special mixer, and heat treatment could be utilized to obtain required strength. Ductal is marketed form of UHPC that developed by participation of three companies; (Lafarge, Bouygues, and Rhodia). Powder’s quartz has (10µm diameter) is utilized in premix UHPC as fillers material, premix furthermore includes steel fibers 2600MPa tensile strength (Schmidt et al. 2012) and (Graybeal 2013) [37 and 38]. Utilizing of these materials increases premix's cost. UHPC commercially available about (65times) more costly of conventional concrete that nearly (60$/m^3). UHPC's cost involves material's cost of admixture, and delivery, shipping transportation, and so on.

1-8 High Performance Concrete (HPC) definition

Materials of construction have various terms in various countries. In verity, world of today is rather small. We're utilizing quite similar materials in different world's parts even although they might have various names. High performance refers to material's external characteristics [31]. Federal High Way Administration FHWA [39], defines HPC by eight characteristics: abrasion resistance, freeze thaw durability, chloride penetration, scaling (volumetric change) resistance, modulus of elasticity, compressive strength, creep, and shrinkage. The term (high performance) can related to each kind of concrete that offers fresh or hardened characteristics surpass those of conventional concrete.

1-9 Structures' Performance criteria

For structures of today, we're looking for material have four discriminatory characteristics: (strength, durability, workability, and affordability), the first three characteristics essentially comprise whole eight requirements of performance mentioned above, affordability means cost, when says high performance, refers to refinement in some or whole of those characteristics. Occasionally, one's has to give-up a little in the one to a little gain in other. Those four properties as following:

1-9-1 Strength

Higher strength offers material saving. Structural dead load, or weight, are major loading in structures' designing. Thus, higher strength generally gives two main advantages: less weight and less material, the weight decreasing leads to reduce material demand, because it reduces the loads the structures have to carry. With strength of (200MPa) UHPC is nearly such as steel, excepting the tensile capacity relatively low, consequently UHPC can't be utilized like steel [31].

1-9-2 Workability

Concrete's workability is board and subjective term describing how easily freshly mixed concrete could be mixed, placed, consolidated and finished with least damage of homogeneity. Workability is peculiarity that directly impacts strength, quality, appearance, and even labors' costs [31].

1-9-3 Durability

The design life of the specified structure is requiring researches on materials those have durable, and reduction in the maintenance efforts [31]. Concrete's durability is impacted by concrete's resistance to fluid penetration. This is mostly affected by w/c and cementitious materials' composition utilized in concrete, for a given w/c, utilize of slag cement, fly ash, silica fume, or mixing of these materials typically will increase the concrete's resistance to fluid penetration and thus promote concrete durability [3]. UHPC shows good possibility in this filed.

1-9-4 Affordability

Affordability is cost, so its control if the material could be utilized or not, even if the material has good characteristics, except that their high cost may had prevent them from being utilized. Stainless steel, for instance is very good for abundant construction implementations. But, its higher cost is obstructing to widespread utilizing [31].

1-10 Application of UHPC

Exceptional technical and economic advantages were obtained by utilizing UHPC. Because of these benefits, UHPC presently being orderly utilized in several implementations; bridges, pavements, and buildings. UHPC is oftentimes utilized doesn't due to its strength, but due to the other engineering characteristics those escorted the high strength, like, increased stiffness (modulus of elasticity), high abrasion resistance or minimize permeability to prejudicial materials. In bridges, UHPC is utilized to obtain one or collection of following mechanical characteristics:

- Increase span length;
- Increases the spacing of girders; and ;
- Reducing thickness of sections.

The decreased permeability of UHPC presents occasions for promoting durability, and increment service life.

In buildings, UHPC presents opportunities for reducing of column's sizes (for example), resulting in lower concrete's volumes, and large reductions in dead loads. Adding UHPC's layers to existent members to structure's upgrade [31], Table (1-1) and (Fig. 1-8), shows examples of UHPC applications around word (Azmee et al. 2018) [40].

Table (1-1) UHPC application around the world [40].

Structures applications	Location	Completion production year	Compressive strength MPa	Flexural strength MPa
Sherbrooke footbridge	Sherbrooke Canada	1997	200	40
Bourg les Valence	France	2005	---	---
Joppa clinker silo	Illinois USA	2001	220	50
Seonyu footbridge	Seoul Korea	2002	180	32
Sakata Mirai footbridge	Sakata Japan	2002	238	40
Millau Viaduct toll gate	A75 Motorway-France	2004	165	30
Shepherds creek bridge	Sydney Australia	2005	180	---
Blast resisting panels	Melbourne Australia	2005	160	30
Mars Hill Bridge Road bridge	USA	2006	---	---
Papatoetoe footbridge	Auckland New Zealand	2006	160	30
Glenmore Legs by bridge	Calgary Canada	2007	---	---
Gaertnerplatz bridge	Kassel Germany	2007	150	35
UHPC girder bridge	Iowa USA	2008	150	---
Wind turbine foundations	Denmark	2008	210	24
Mackenzie River twin bridges	Thunder Bay Canada	2011	120-200	15-40
Haneda Airport slabs	Tokyo Japan	2010	210	45
Whiteman Creek bridge	Brantford Canada	2011	140	30
Sewer pipes	Germany	2012	151	---
Spun concrete columns	Germany	2012	179	---
UHPC truss footbridge	Spain	2012	150	---
Haneda Airport	Tokyo Japan	2010	---	---
The Rotman School of Management Expansion	Toronto Canada	2012	---	---

Museum of European & Mediterranean Civilizations	Marseille France	2013	---	---
Cap Cinéma	Rodez France	---	---	---
Renovation of a Pool	Amiens France	---	---	---
Chukuni River Bridge	Ontario Canada	2010	---	---
stadium Jean Bouin, Roof & Facade	Paris	2013	---	---

Fig. 1-8 UHPC's applications around the world [40].

Continue to Fig. 1-8 UHPC's applications around the world [40].

1-11 Research Objectives

The major objective of this study is to investigate near surface CFRP effectiveness in shear behavior of UHPC tapered-beam has multi longitudinal openings. The experimental program was carried out:

- Explore effect of longitudinal openings and their locations on shear capacity for UHPC tapered-beams;
- Study the effect of CFRP bars and CFRP strips orientations on shear behaviour for UHPC tapered-beams;
- Study effect of inclination angle of UHPC tapered-beams on its shear capacity;
- Confirm influence of some main factors such shear span to effective depth a/d ratio, and tensile bars ratio on shear strength of tested UHPC tapered-beams;
- Investigate UHPC tapered-beams' shear behavior up to failure with and without stirrups;
- Investigate steel fiber effect of UHPC tapered-beams on its shear capacity;

- Making comparison between the strengthening methods NSCFRP bars, CFRP strips, stirrups, and steel fibers, and finally;
- Check the safety level and efficiency of 19 proposed equations shear flexural design in the codes to predicting the UHPC tapered-beams' shear strength with and without transverse bars.

1-12 Outline of Dissertation

Chapter One: Contains features of general introduction, study's objective, and outline of dissertation.

Chapter Two: Presents background, shear behaviour FRP, and UHPC literature review.

Chapter Three: Describes experimental work of design of concrete mix, details of tested concrete tapered-beams, procedure of (mixing, casting, and curing), and strengthen by CFRP bars/strips techniques with materials characteristics (CFRP, steel reinforcement, adhesive material, silica fume, fine aggregate, cement, steel fibers, and superplasticizer).

Chapter Four: Shows experimental testing results of tapered-beams were mainly investigated with all reading and recorded during testing in the sense of cracks, deflection, etc., and shows the results' discussion.

Chapter Five: Provides summary for this research, and its conclusions with recommendations.

Finally, whole references those gave help during this study were presented.

CHAPTER TWO

LITERATURE REVIEW

2-1 GENERAL

UHPC is advanced cementitious materials with outstanding material properties, composed of water, additives, aggregates, silica fume, cement, fibers and admixtures.

The distinction between conventional concrete and UHPC mix design is; aggregate size, particular in the binder amount, and the fibers presence. Utilize of super-plasticizer in quite amount in order to acquire a reasonable workability is likewise another UHPC characteristic, comparing with conventional concrete, UHPC matrix is extremely denser, self-consolidating, low permeability, and very high mechanical characteristics, 150 MPa to 250MPa compressive strength, and 7MPa to 15MPa uniaxial tension. Moreover, its intense matrix produces minimal permeability, consequently promoted durability is expectancy [35].

Development of Ultra High Performance Reinforced Concrete UHPFRC originated in (1970s) by Yudenfreund et al. [41] who explored high strength cement pastes had low w/c ratio.

Hans Hendrik Bache was developed a material with high fiber content [32]. That material was called Compact Reinforced Concrete CRC the first information about this were published in (1981), this construction special form is still utilized frequently today, essentially for balconies, and stairs primarily in Denmark, Bache’s idea was taken up in (1994) by the French contractor Bouygues [35] and developed further, and by cooperating with (Lafarge), a new concrete mix was devised Reactive Powder Concrete RPC, which exists in the form of Ductal. One early application is involved replacing of steel beams by UHPC ones in the cooling towers of power station at Cattenom in France, the steel beams were having to replace because they're corroding in the highly aggressive environment inside cooling towers.

Different UHPC products were developed over last few years, UHPC is gaining increased heeding in several countries with utilize it in bridges, building, repair and rehabilitation, architectural features, columnar components like towers (oil, gas, windmills), hydraulic structures, off-shore structures, and overlay materials. The utilize of UHPC for bridges, and bridge components can be seen in different countries like: France, Japan, Germany, Denmark, Australia, China, Italy, Austria, Canada, Malaysia, Czech Republic, Netherlands, Slovenia, Korea, Switzerland, New Zealand, and United State (AItcin et al. 1998) [42]. But in Iraq UHPC hasn't utilized yet.

UHPC with high compressive strength, and durability improving exemplify a concrete technology's quantum leap. UHP material is offer interesting implementations' diversity. It allows the economic buildings and sustainable to constructed with an exceptional slender design. Its ductility and strength make it definitive building material. Besides that, its distinguished resistance contra all corrosion's types are an extra milestone towards constructions with no-maintenance. UHPC has extremely special characteristics those are remarkably various to the characteristics of HPC and normal concrete. For completely utilization of UHPC's superior characteristics, distinguished knowledge is desired for producing, designing and constructing. Worldwide UHPC is under itemized exploration (Aaleti et al. 2017) [43]. Many elements structural or constructions were already constructed with UHPC utilizing.

The maintenance and strengthening of the construction members is very important for the old constructions for the purpose of increasing its design life, and the maintenance sometimes more economical than the demolition and construction again. UHPC were firstly developed in France, at the end of the last century, Ductal has been deployed on a vast array of projects internationally. Lafarge teams have developed a unique level of experience.

This chapter provides a brief overview of some literature review of international recommendations those were deal with UHPC, mix design, shearing strengthening, and concrete members those have openings.

2-2 UHPC's characteristics

UHPC tend to consist of high contents of cementitious materials, low w/b or w/c ratios, high compressive strength, and high tensile strength resulted from fiber reinforcement. In particular important, UHPC doesn't show micro-cracking in early age that usually occurs in conventional concrete. This merit, combined with homogeneous matrix of cementitious, results in the concrete with in the extreme low permeability. The response of UHPC's tensile mechanical also exceed that of normal concrete. Steel fibers those included in UHPC components permit concrete to preserve tensile capacity of UHPC beyond the cracking of cementitious matrix. UHPC sustained tensile capacity, and its durability, present occasions to rethink common concepts in the design of reinforced concrete structure, e.g. UHPC tensile capacity can eliminate the needing of steel reinforcement for some of structural members. And UHPC's durability could reducing reinforcement cover (Graybeal 2012) [44].

2-2-1 UHPC's compressive stress strain behaviour

UHPC has linear relationship of stress strain till failure. In the extreme brittle failure would noticed.

(AFGC) [29] had insert UHPC recommendation and presented shape of stress strain relationship (Fig. 2-1).

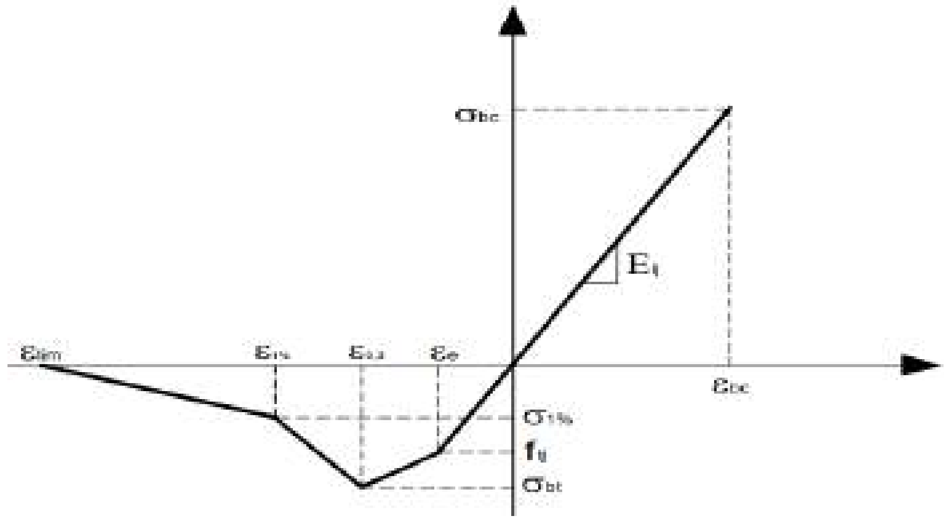


Fig. 2-1 AFGC's Stress strain curve [29].

(Japan Society of Civil Engineering JSCE 2004) [45] presented idealize stress strain curve this may ordinarily use for ultimate limit state evaluation for members subjected to the axial and/or flexural force as in (Fig. 2-2).

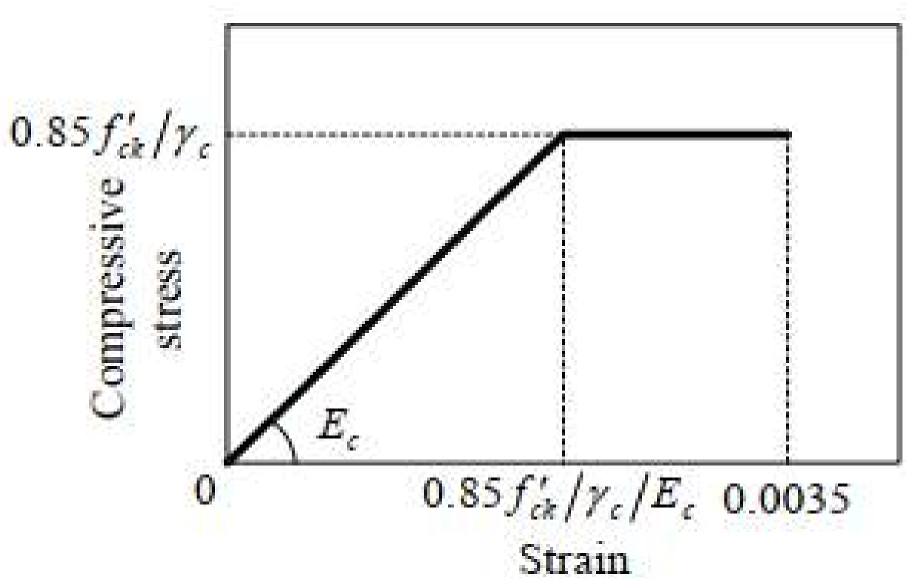


Fig. 2-2 JSCE Stress strain curve [45].

(Graybeal 2006) [46] introduced exponential equation has two constants (a, b) as a modification factors to modify any deviation of the actual behaviour of the compressive stress strain curve from linear one (Fig. 2-3). This equation derived to be appropriate in the ascending branch of the curve. Thus, the relationship between stress strain curve will be as:

$$f_c = \varepsilon_c E (1 - \alpha) \qquad .....(1)$$

$$\alpha = a\, e^{(\varepsilon c\, E / fc\, b)} - a \qquad .....(2)$$

The values of the (a, b) constants are illustrative in the Table (2-1).

Table (2-1) Equation’s two constants value.

Curing Regime	a	b
Steam	0.001	0.243
Untreated	0.0114	0.440
Tempered steam	0.0041	0.341
Delayed steam	0.0044	0.358

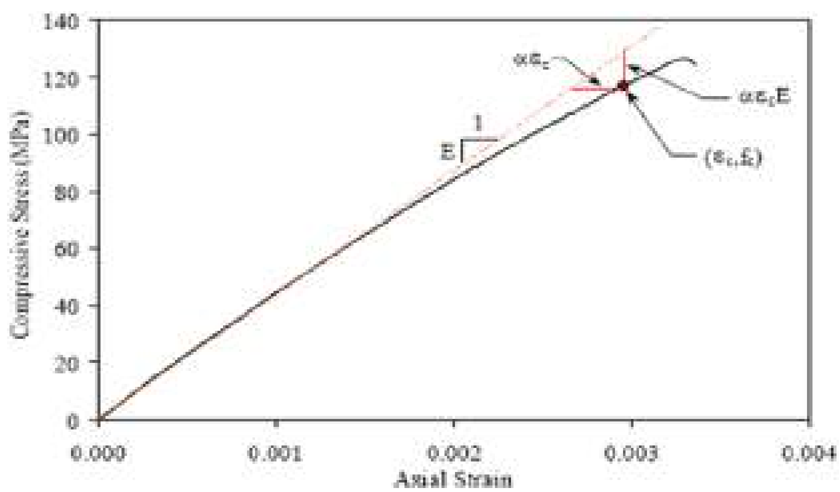


Fig. 2-3 Stress strain curve [46].

2-2-2 UHPC’s tensile behaviour

Below is a brief on tensile stress as mentioned in some of the codes and recommendations those available:

(AFGC) [29] 8 MPa is the design recommended value for tensile stress.

(Australian design guidelines) [47] the tensile stress could be taken as 8 MPa.

(JSCE) [45] considering the safety margin has recommended to utilize 8 MPa as tensile stress value.

The UHPC's tensile strength is approximately 5% from compressive strength [46] in the whole of curing regimes those mentioned in the Table (2-2). In same time has mentioned other consideration that is an equation for tensile strength (psi-units) as below:

$$f_{ct} = x \sqrt{fc'} \qquad \text{.....(3)}$$

Table (2-2) shows x-values.

Table (2-2) The value of constant for equation's 3.

Curing regime	*x*
Steam treatment	*7.8*
Un-treated	*6.7*
Tempered-Steam	*8.3*
Delayed steam-treated	*8.3*

2-2-3 UHPC's modulus of elasticity (Ec)

Because cylinders compression testing is an extremely utilized quality control method QCM for the structural concrete, therefore engineers often endeavor to link the other characteristics of the concrete's behaviour to the results of compression test. Numerous of the researchers and international codes have developed works those focuses on relation between the concrete compressive strength and its elastic modulus. Table (2-3) is totalize of recommendations of UHPC's Ec.

Table (2-3) Connection between E_c and UHPC compressive strength.

Researcher (s) or Committee	Equation	Note
(Kakizaki et al. 1992) [48]	$E_c = 3650\sqrt{f_c'}$ GPa	f_c' from 83 to 138
(Popovics et al. 1998) [49]	$E_c = 9500\ (f\ c'+8)^{0.33}$ GPa	
[47]	50 GPa	
AFGC [29]	55 GPa	
JSCE [45]	50 GPa	
(Ma et al.) [50]	$E_c = 19000\ (f_c' / 10)^{1/3}$ GPa	without coarse aggregate
(Graybeal 2012) [51]	$E_c = 4069\sqrt{f_c'}$ psi	$97 \leq \sqrt{f_c'} \leq 179$
(ACI 318-14) [3]	$E_c = 0.043\ \rho^{1.5} * \sqrt{f_c'}$ psi $E_c = 5311\ \sqrt{f_c'}$ psi	ρ = concrete unit weight = 2480 kg/m³

2-3 The effect of heat treatment HT regime curing on UHPC

In general, heat or steam treatment improves many properties of UHPC. These improvements cannot be obtained without utilizing of heat treatment. Below is an overview of some of the recommendations in this regard:

The components those HT had reach their final maturity and could therefore to be utilized with no need to waiting (28-days) or more compared with the conventional concretes, after HT the tensile and compressive strengths about 10% higher than (28-days) strength with normal water storage, the total shrinkage will be zero after HT, significantly reducing the Creep; the creep's coefficient will be 0.2 instead of 0.8 without HT, and the durability is get better as result of the reduction in voids ratio [29].

To boost the high strength and high density, contributed in reduction in creep and shrinkage, and improving durability, UHPC necessitates in principle 48-hours heat curing at 90℃. Most of the all UHPC's shrinkage occurs during heat curing around 450 micro-strain, while only around 50 micro-strain is remain after heat curing [45].

Compressive strength of UHPC increases by steam treatment, increases UHPC's modulus of elasticity, decreases UHPC's creep's coefficient from 0.78 to 0.29, and substantially exclude long term shrinkage. Also decreases the penetrability of chloride ion to negligible level, and significantly UHPC's enhances abrasion resistance [46].

Heat treatment, and high homogeneity of materials due to the utilizing a quite fine aggregate (sand) only, participate to eliminated initiation of extensive the early age cracks those're UHPC's major disadvantage. Those lead to get superior UHPC's mechanical properties, like very high tensile and compressive strengths, high ductility, high modulus of elasticity, and fatigue strength will be high too [30].

2-4 Flexural design of UHPC

There are some international recommendations, and some researchers are suggested equations for estimation of section's capacity in UHPC sections. The variation between the equations is the simplification of stress block of compression and tension, some equations facilitate the actual stress block to a rectangle stress block and the others facilitate it to a triangle and a square stress block.

(Nasser 2016) [24] has derived a theoretical equations based on JSCE [45] recommendation simplify genuine stress block, (Fig. 2-4).

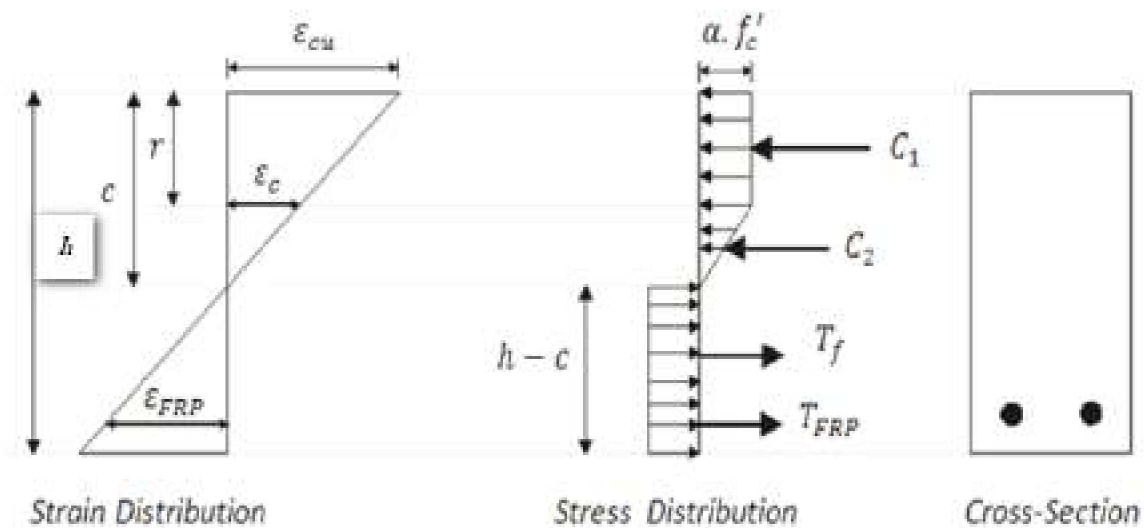


Fig. 2-4 Distribution of stress strain [24].

$C_1 = \propto. f_c'. b. r$, $r = \left[1 - \frac{\propto. f_c'.}{\varepsilon_{cu}.E_c}\right] * c$, $c = \frac{\varepsilon_{cu}}{(\varepsilon_{cu}+\frac{f_{FRP}}{E_{FRP}}}$, sub the eqs. of (r & c) in eq. of C_1 then;

$$C_1 = \propto. f_c'. b. \left[1 - \frac{\propto. f_c'}{\varepsilon_{cu}.E_c}\right] * \frac{\varepsilon_{cu}}{\varepsilon_{cu}+\frac{f_{FRP}}{E_{FRP}}} * d \qquad \text{....(4)}$$

$C_2 = \frac{1}{2}(c-r) * \propto. f_c'. b,$ sub the value of (r) the eq. of C_2 will be;

$$C_2 = \frac{1}{2} * \frac{(\propto. f_c')^2}{\varepsilon_{cu}.E_c}. b. \frac{\varepsilon_{cu}}{(\varepsilon_{cu}+f_{FRP})} * d \qquad \text{....(5)}$$

$T_f = 0.4 * \sqrt{f_c'} * b. (h-c)$, after sub the (c) eq. in the T_f eq. will be;

$$T_f = 0.4 * \sqrt{f_c'}. b. [h - \frac{\varepsilon_{cu}.d}{\left(\varepsilon_{cu}+\frac{f_{FRP}}{E_{FRP}}\right)}] \qquad \text{....(6)}$$

$$T_{FRP} = A_{FRP} \times f_{FRP} \qquad \text{....(7)}$$

$(\sum T = \sum C)$

$$\propto. f_c'. b. \left[1 - \frac{\propto. f_c'}{\varepsilon_{cu}.E_c}\right]. \frac{\varepsilon_{cu}.d}{\left(\varepsilon_{cu}+\frac{f_{FRP}}{E_{FRP}}\right)} + \frac{1}{2}. \frac{(\propto. f_c')^2}{\varepsilon_{cu}.E_c}. b. \frac{\varepsilon_{cu}}{\left(\varepsilon_{cu}+\frac{f_{FRP}}{E_{FRP}}\right)}. d = A_{FRP} * f_{FRP} +$$

$0.4 * \sqrt{f_c'}. b. [h - \frac{\varepsilon_{cu}}{\left(\varepsilon_{cu}+\frac{f_{FRP}}{E_{FRP}}\right)}. d]$ By Multiplying $(\varepsilon_{cu} + \frac{f_{FRP}}{E_{FRP}})$ so,

$\propto . f_c^{'} . b . \varepsilon_{cu} . d . \left(1 - \frac{\propto . f_c^{'}}{\varepsilon_{cu} . E_c}\right) + \frac{1}{2} . \frac{(\propto . f_c^{'})^2}{E_c} . b . d = 0.4 . \sqrt{f_c^{'}} . b . h . \left(\varepsilon_{cu} . \frac{f_{FRP}}{E_{FRP}}\right) -$
$0.4 . \sqrt{f_c^{'}} . b . \varepsilon_{cu} . d + A_{FRP} . f_{FRP} . \left(\varepsilon_{cu} + \frac{f_{FRP}}{E_{FRP}}\right)$ after simplified the eq. get;

$$\propto f_c^{'} b \varepsilon_{cu} d \left(1 - \frac{\propto . f_c^{'}}{\varepsilon_{cu} . E_c}\right) + \frac{1}{2} \frac{(\propto . f_c^{'})^2}{E_c} bd = 0.4\sqrt{f_c^{'}} bh \, \varepsilon_{cu} + 0.4\sqrt{f_c^{'}} bh \frac{f_{FRP}}{E_{FRP}} - 0.4\sqrt{f_c^{'}} b \varepsilon_{cu} d + A_{FRP} f_{FRP} . \varepsilon_{cu} + A_{FRP} \, \varepsilon_{cu} \frac{f^2{}_{FRp}}{E_f} \quad \text{....(8)}$$

$$M_n = C_1 \left(c - \frac{r}{2}\right) + \frac{2}{3} * C_2 (c - \mathrm{r}) + \frac{1}{2} T_f (h - c) + f_{FRP} A_{FRP} (d - c) \quad \text{....(9)}$$

2-5 Shear strength of tapered-beam

The first researcher who is tested tapered-beam was (Mörsch 1922) [52]. The tested beams design are illustrative in (Fig. 2-5).

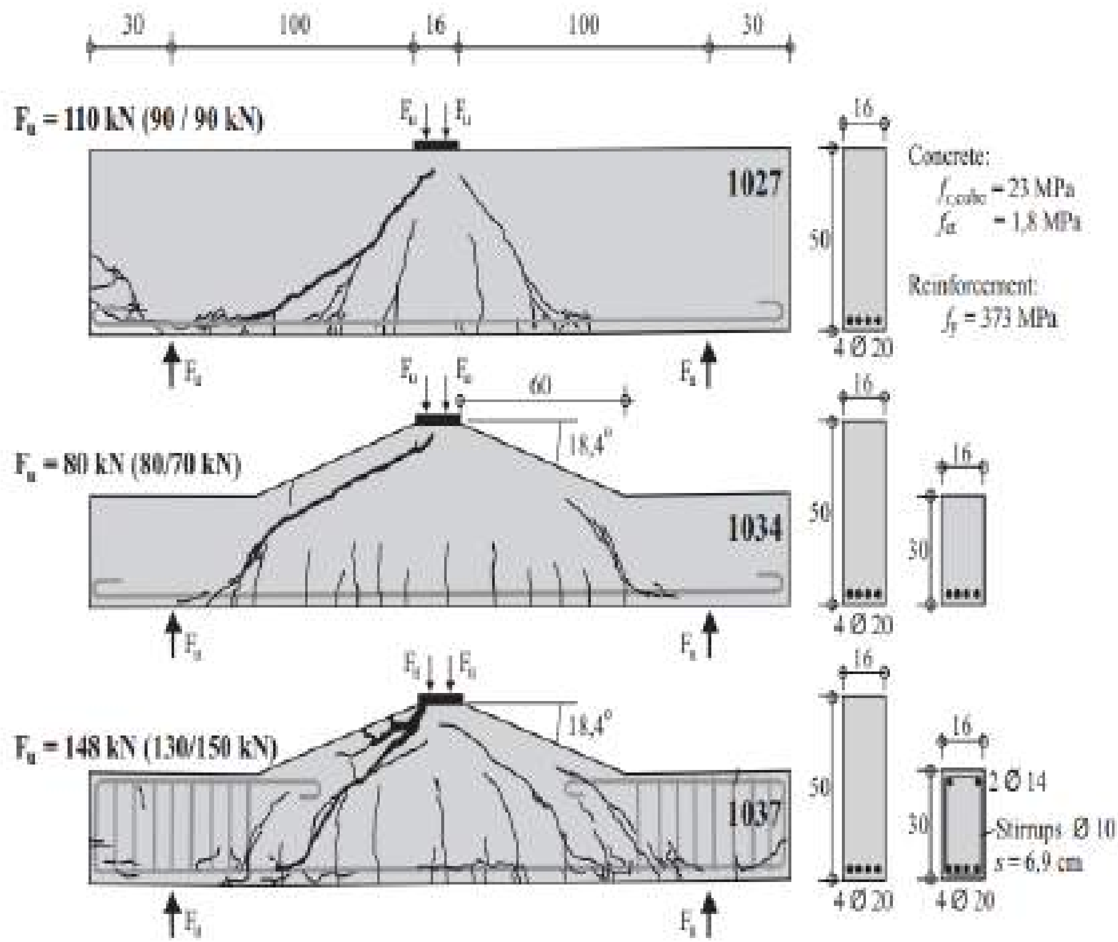


Fig. 2-5 Tested beams utilized by [52].

Appeared tapered-beam with number 1034 and inclination angle 18.4˚ and hasn't transverse bars, had load capacity 20% lower than prismatic beam with number 1027. The tapered-beam with number 1037 that has transverse bars in support's zone showed considerable greater load capacity than tapered-beam number 1034, (Fig. 2-5). Knowing that the bars utilized were plain bars not deformed ones.

The (ACI 1973) [53] after exploring available great number of results experimental, presented a formula to calculate ultimate shear, as follows:

$$V_{Rd} = 0.17 \sqrt{f_c'}\, b\, d \qquad ....(10)$$

(DIN1045-1) [54], (Fig. 2-6) mentioned the accounting necessity for the component of shear of compression force and inclined tensile rebar of concrete in members have variable depth, there aren't any indications of the tapered-beams critical section that where shear capacity must be calculated.

Since the depth of tapered-beam varies along its axis from middle portion to support, it almost hasn't any practical sense if wasn't specified critical section. As a result, the structural designer engineers are generally utilizing such members based on experiential background. So as to ensure sensible design, over and above to understand shear behaviour of tapered-beams, it's necessary to probe the mechanism of shear resistance of tapered-beams. The shear design formula for tapered-beam is introduced as follows:

$$V_{Ed} = V_{Ed0} - V_{ccd} - V_{td} - V_{pd} \leq V^{a}_{Rd} \qquad ....(11)$$

Where:

V_{Ed0}: Shear force due to dead loads and live loads;

V_{ccd}: Design shear resistance due to inclination of compression chord of beam;

V_{td}: Design shear resistance component of inclined longitudinal tension reinforcement;

V_{pd}: Design shear resistance component of prestressed force;

V^{a}_{Rd}: Design value of shear bearing capacity of tapered-beam at design section.

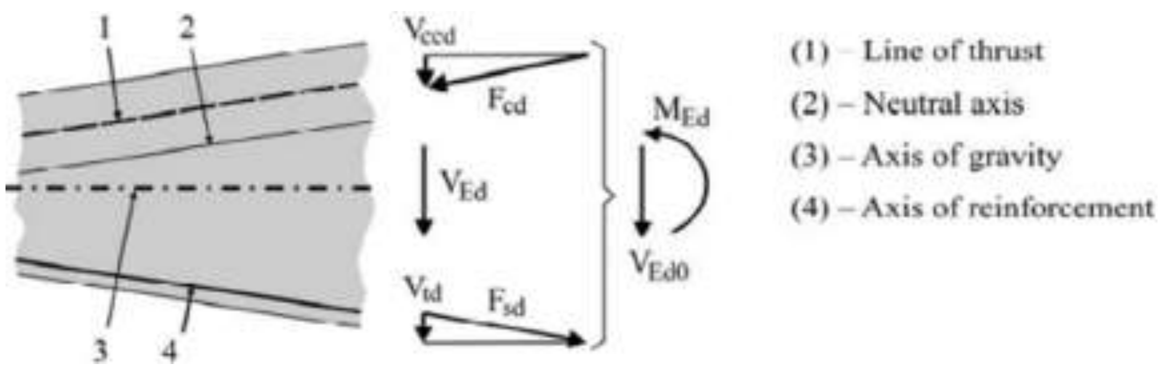


Fig. 2-6 Shear resistance components of varied depth concrete member [54].

If member without (pre-stressing, and horizontal tensile bars), where (V_{pd} and V_{td}) are amounting to zero, the designing shear formula be:

$$V_{Ed} = V_{Ed0} - V_{ccd} \leq V^{a}_{Rd} \quad ....(12)$$

The value of V_{ccd} defines as;

$$V_{ccd} = (M_{ed} / Z) \tan\alpha \quad ....(13)$$

(AFGC) [29], divided the shear capacity into three components as equation below:

$$V_u = V_{Rb} + V_a + V_f \quad ....(14)$$

Where;

V_{Rb} : Concrete participation;

V_a : Reinforcement participation;

V_f : Fibers participation.

- For reinforced concrete:

$$V_{Rb} = 0.21\ k \sqrt{f_c'}\ b\ d / (\gamma E\ \gamma b) \quad ....(15)$$

- In compression:

$$k = 1 + (3\ \sigma_m / f_c') \quad ....(16)$$

- In tension:

$$k = 1 - (0.7\ \sigma_m / f_c') \quad ....(17)$$

σ_m: Mean stress in concrete total section under normal design force.

- For pre-stressed concrete:

$$V_{Rb} = 0.24 \sqrt{f_c'}\ b\ z / (\gamma E\ \gamma b) \quad ....(18)$$

In absence of longitudinal passive or pre-stressing reinforcement, those terms reduced to minimum value at which shear cracks appear, with appropriate safety margin. γE= safety coefficient:

$$\gamma E * \gamma b = 1.5 \quad ....(19)$$

$$Va = 0.9\ d\ (At\ Fe / St\ \gamma s)\ (\cos\alpha + \sin\alpha) \quad ....(20);\ \text{or}$$

$$Z\ (At\ Fe / St\ \gamma s)\ (\sin(\alpha+\beta)/\sin\beta) \quad ....(21)$$

$$Vf = S\ \sigma_p / (\gamma b\ f\ \tan\beta_u) \quad ....(22)$$

Where:

S: Fiber's effect area, estimated as; [$S=0.8*(0.9*d)^2$ or $(0.8*z^2)$] for circular sections, and [$S=(0.9*b*d)or(b*z)$] for rectangular or T-sections.

σp: Residual tensile strength,

$$\sigma_p = \frac{1}{k*Wlim}\int_0^{Wlim} \sigma(W)dw \qquad \text{....(23)}$$

With, $W_{lim}=max\ (w_u,\ 0.3mm)$ and $(Wu=lc*\varepsilon_u)$ $[\sigma(w)]$ experimental characteristic post cracking stress for crack width (w), (W_u) ultimate crack width, i.e. value that attained at (ULS) for resistance to combined stresses, on outer fiber, under moment exerted in section, (K) is orientation coefficient for general effects.

βu = Angle of compression struts with lower bounded value of (30°).

The design shear strength (V_{yd}) for UHPC that developed by JSCE [45] could be obtained by the following equation:

$$V_{yd} = V_{rpcd} + V_{fd} + V_{ped} \qquad \text{....(24)}$$

Where, V_{rpcd} is the concrete shear capacity, and its equation is:

$$V_{rpcd} = 0.18\sqrt{f_c'}\ b_w\ d/\gamma_b \qquad \text{....(25)}$$

f_c'= UHPC design compressive strength;

γ_b = 1.3 safety factor;

$V_{fd}=(F_{vd}/tanB_u)b_w\ Z/\gamma_b$ (design capacity that provided by steel fiber); ...(26)

F_{vd} = UHPC tensile strength;

B_u = diagonal crack inclination ≥ 30˚, $Z = 0.9\ d$ (lever arm),

V_{ped} = Component of the tendons effective tensile force (parallel to the shear force).

Due to the inclination of longitudinal reinforcement (Albegmprli et al. 2018) [55] are proposed model for inclined flexural reinforcement contribution in shear capacity, thus there must be add another term for the equation No.24 for the dowel action effect, and equation 4 will be:

$$V_{yd} = V_{rpcd} + V_{fd} + V_{ped} + V_{da} \quad \text{....(27)}$$

V_{da} = Dowel action of inclined main reinforcement, and its equal to:

$$V_{da} = 0.2\, A_s\, f_y\, \sin\theta \quad \text{....(28)} \quad [55]$$

In (ACI 318-14) [3], the term "effect of inclined flexural compression" is utilized for explanation the distribution of various stresses of tapered-beam in comparing with prismatic beam. The results of stress allocation in shear resistances forces as columnar component diagonal flexural stress. Those data were find to be too cursory to apply simply in designing of tapered-beams.

2-6 Longitudinal opening

The longitudinal openings are used to constructed a hollow core beams are cast in the site, pre cast or pre-stressed concrete members with continuous void has provided to minimize weight, cost, and also to use it for mechanical, sewage, water, and hidden electrical runs.

(Hadi 2013) [56], casted five RC tapered-beams. They had two shapes of hollows circular and square. Beams dimensions were, (1.17m, 0.26m, and 0.15m) which tested by two point loads. The purpose of the research was to get a tapered beam with hollow similar to the solid beam in terms of capacity. The parameters were type of section (hollow/solid), shape, and materials of hollow (plastic/iron pipe). (Fig. 2-7). He concluded that the hollows caused decreasing in the sections' stiffness and increase its strain and deflection.

When increasing of the ratio of shear reinforcement the section loads capacity increase too, but the deflections were decrease. Also, he got the capacity of tapered hollow beam (square hollow) greater than the solid beam.

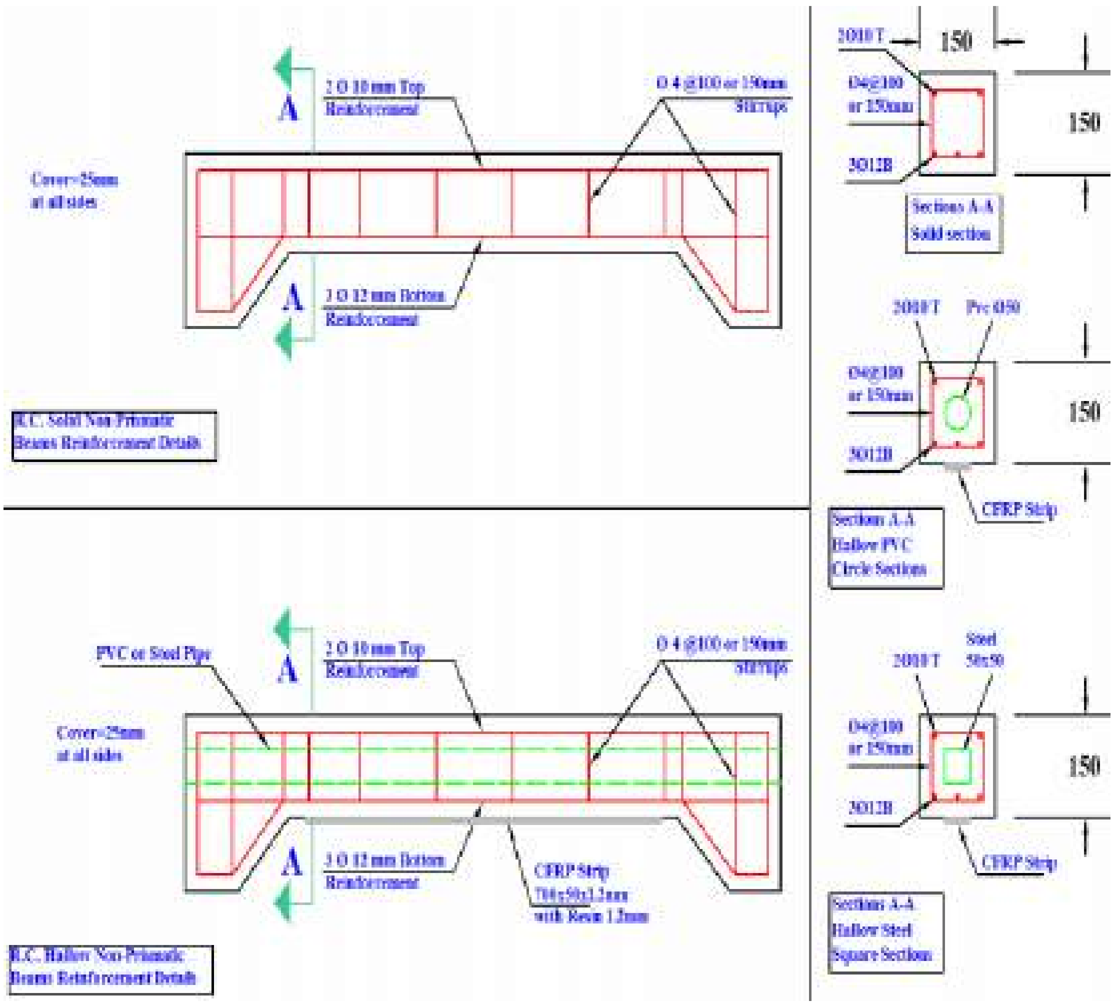


Fig. 2-7 Hadi’s parameters [56].

(Ahmad et al. 2014) [57] they investigated the behaviour of six (solid and with opening) beams with dimensions (length 1m, height 0.18m, width 0.12m), simply support. The tested load was (partial uniformly distributed). Four beams were containing longitudinal opening with varied section (80mm x 40mm) and (40mm x 40mm). Their parameters were, size of opening, stirrups effect, and stirrups orientation (Fig. 2-8). The result is showed the existence of hollows are reduction the loads carrying capacity, and increased the deflections, stirrups are decreased whole deformations at whole phases of loading, especially after the initial cracking, and the ductility is raised when the hollow ratio reduced or stirrups increased by about 50%.

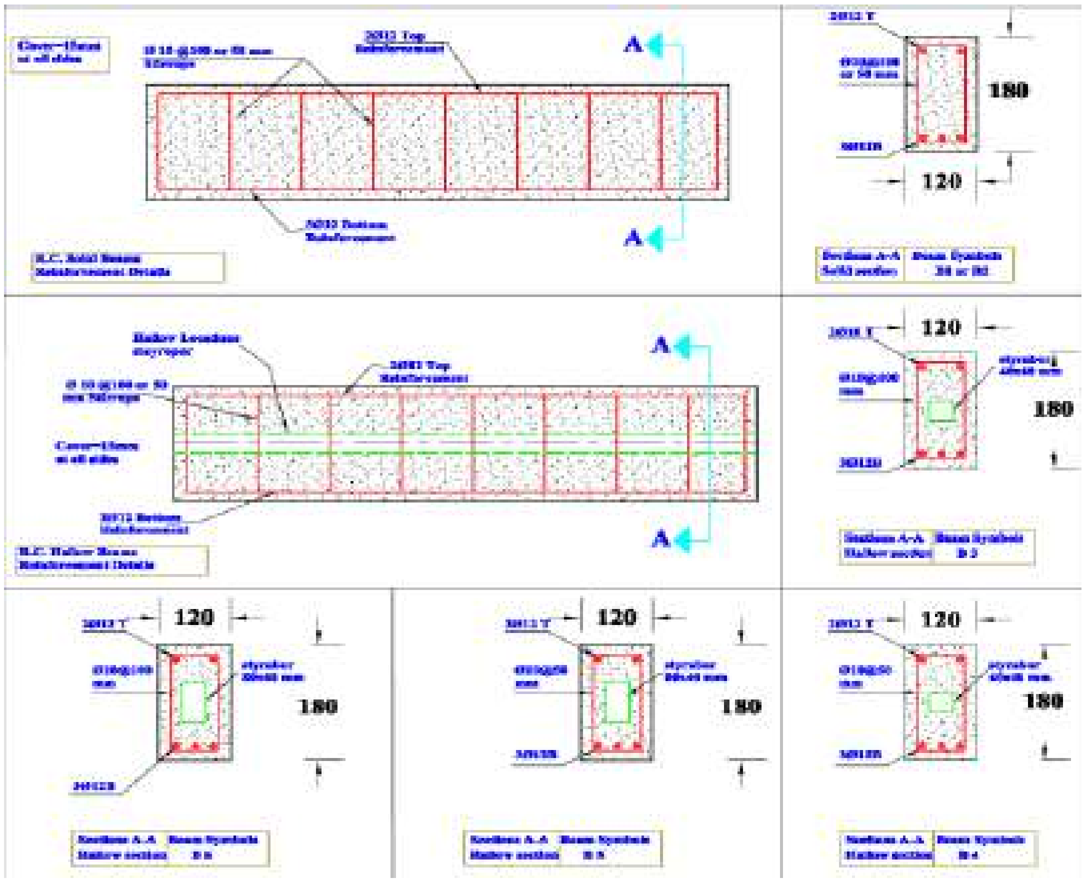


Fig. 2-8 Parameters of Ahmad et al. [57].

(Murugesan et al. 2016) [58] they are casted thirteen beams (1.7m, 0.15m, 0.25m) to investigate flexural strength of simply supported hollow beams with longitudinal circular hole made by utilized PVC pipe are tested by applied two point loads. Each hollow had (25mm, 40mm, or 50mm) in diameter, with variable hollow's center location position (45mm to 180mm) from beam's top. The parameters were; the size and location of the hole, (Fig. 2-9). The results were, increasing of hole's size resulted in decreasing in the both first cracking and ultimate load, the section capacity will decrease when the hollow located in stress block, the section capacity of beam has hollow located under stress block was higher than others hollow beams.

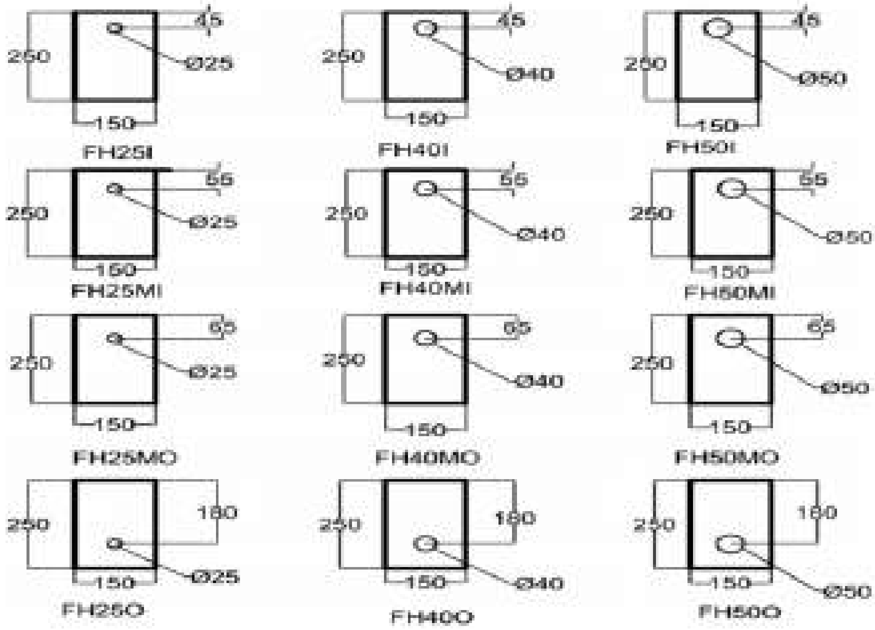


Fig. 2-9 Parameters of Murugesan et al. [58].

2-7 Significant factors for shear bearing capacity

Experimental and analytical studies had exposed that elements' shear capacity is ruled by next hegemony factors: 1- Longitudinal reinforcement (ρl ratio), 2- Shear span to depth (a/d ratio), 3- Size effect (d), 4- Concrete's strength (f'_c), and 5- Axial force (ASCE-ACI 445) [59]. Nevertheless, the importance of every parameter to elements' shear capacity are yet under discussion. The importance of those parameters and several views of their participation to elements' shear strengths could be epitomized as next:

2-7-1 Effect of shear span to effective depth a/d & stirrups

As it known the bearing capacity of beam depends on some conditions one of these is shear span to effective depth a/d ratio. This ratio is very important because its control which type of failure will happen. For that many researches were done to study a/d effect on concrete beams.

(Dhaiban 2015) [60] he was investigated behavior and performance of non-prismatic HSRC beams in shear. He has casted thirteen beams twelve were nonprismatic, and one was prismatic. His parameters were three (a/d, $f c'$, and beam's shape) as illustrate in (Table 2-4). All beams were tested by two point loads. All specimens were failed by diagonal shear. (Fig. 2-10). His result was a/d ratio's increasing leads to average decreasing in ultimate shear capacity.

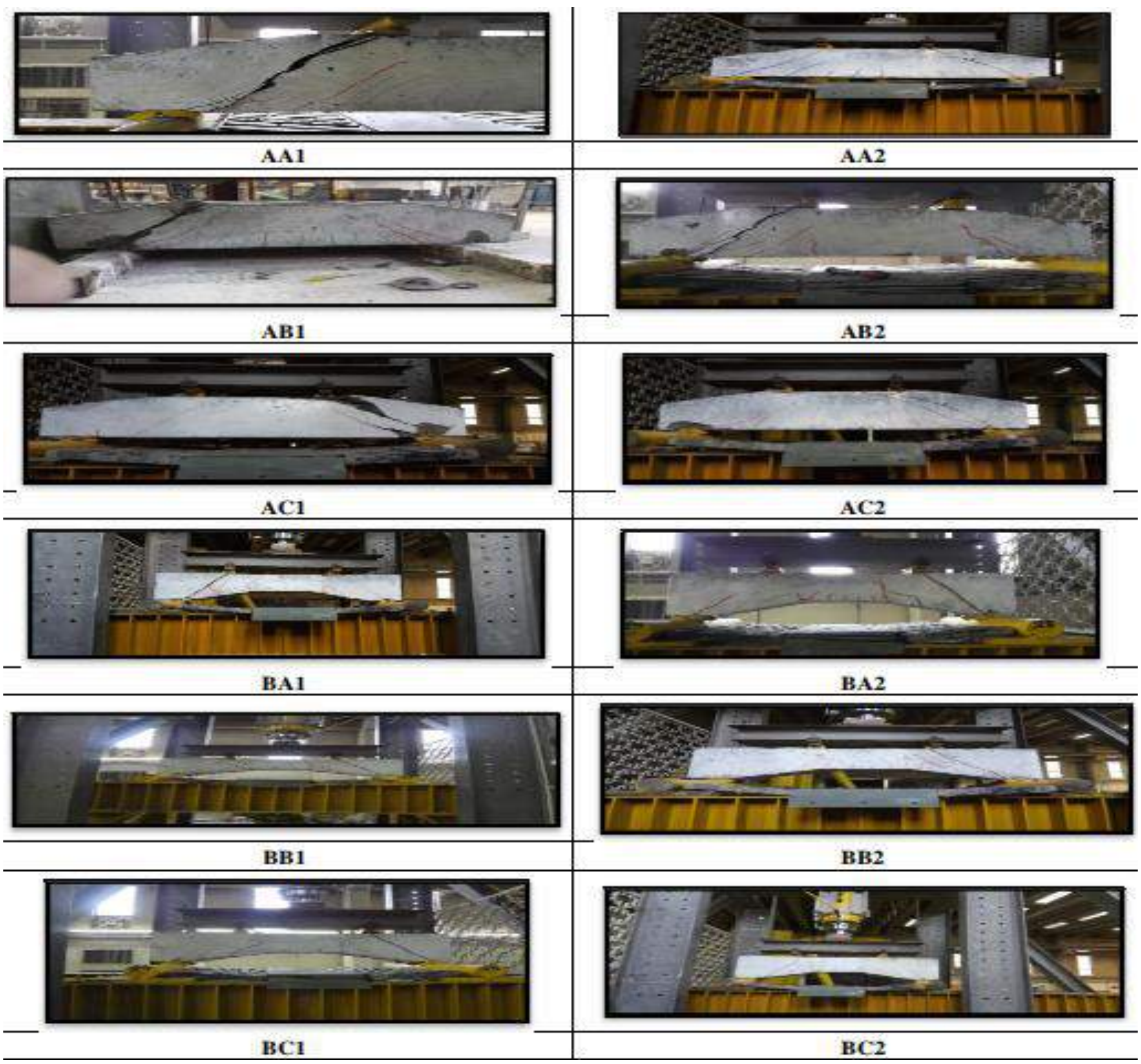


Fig. 2-10 Dhaiban’s specimens pattern failure [60].

Table (2-4) Dhaiban's parameters [60].

Beam designation		Compressive strength (Mpa)	a/d Ratio
AA1		36	1.5
AA2		36	2.5
AB1		54	1.5
AB2		54	2.5
AC1		69	1.5
AC2		69	2.5
BA1		36	1.5
BA2		36	2.5
BB1		54	1.5
BB2		54	2.5
BC1		69	1.5
BC2		69	2.5
CC1		69	1.5

(Chenwei 2015) [61], he's casted ten specimens to study shear resistance mechanism of the RC haunched beams, his parameters were haunched portion's position, concrete cover thickness, stirrups' existence, and tensile bar arrangement, as illustrate in Table (2-5). He concluded that; when a/d ranged from (2.5 - 4), and tensile bar is bent, with small stirrups ratio, main diagonal cracks started from the changing portion of cross sectional (same as bending position of tensile bar) near loading point proceed along inclined tensile bar and toward loading act. When stirrup is provided in RCHBs, more shear and flexural cracks occur. The stirrups number contributing in shear are different. So, the stirrups carried the shear are varying according to diagonal crack's angle and position of tensile rebar close to loading point.

Table (2-5) Chenwei's parameters [61].

<table>
<tr><th>Series</th><th>Specimen</th><th>f_c' (N/mm²)</th><th>a (mm)</th><th>b (mm)</th><th>c (mm)</th><th>e (mm)</th><th>D_s (mm)</th><th>d_s (mm)</th><th>D_m (mm)</th><th>d_m (mm)</th><th>a/d_s</th><th>a/d_m</th><th>ρ_w</th></tr>
<tr><td rowspan="4">I</td><td>H-0</td><td>33.0</td><td rowspan="11">650</td><td>0</td><td rowspan="11">250</td><td>400</td><td rowspan="11">300</td><td rowspan="11">250</td><td rowspan="4">250</td><td rowspan="11">200</td><td rowspan="11">2.6</td><td rowspan="11">3.25</td><td rowspan="5">-</td></tr>
<tr><td>H-100</td><td>33.6</td><td>100</td><td>300</td></tr>
<tr><td>H-200</td><td>29.6</td><td>200</td><td>200</td></tr>
<tr><td>H-300</td><td>36.7</td><td>300</td><td>100</td></tr>
<tr><td>II</td><td>HN-200</td><td>28.6</td><td>200</td><td>200</td><td>300</td></tr>
<tr><td rowspan="3">III</td><td>HS-0</td><td>33.5</td><td>0</td><td>400</td><td rowspan="6">250</td><td rowspan="3">0.314%</td></tr>
<tr><td>HS-100</td><td>28.0</td><td>100</td><td>300</td></tr>
<tr><td>HS-300</td><td>34.4</td><td>300</td><td>100</td></tr>
<tr><td rowspan="2">IV</td><td>HD-100</td><td>34.0</td><td>100</td><td>300</td><td rowspan="2">-</td></tr>
<tr><td>HD-300</td><td>37.4</td><td>300</td><td>100</td></tr>
</table>

f_c': compressive strength of concrete; a: shear span; b: distance between loading point and beginning of haunched portion; c: length of haunched portion; e: distance between support and end of haunched portion; D_s: beam depth at support; d_s: effective depth at support; D_m: beam depth at mid span; d_m: effective depth at mid span; ρ_w: stirrup ratio.

(Chenwei et al. 2017) [62], they are casted seven specimens were divided into three series to studied shear resistance mechanism of prestressed concrete and RC tapered-beams haven't transverse bars. Their parameters were a/d ratio, Table (2-6). Their result was for tapered short beams without stirrups a/d = 1.44, there wasn't taper's effect due to arch actions. And slender RC tapered-beam (2.5< a/d <4.5) had higher shear capacity, while tapered-beam with large a/d ratio (a/d = 5) had smaller shear capacity.

Table (2-6) Chenwei's specimens' details and parameters [62].

<table>
<tr><th>Series</th><th>Name</th><th>α_t (°)</th><th>f_c' (N/mm²)</th><th>a (mm)</th><th>d_s (mm)</th><th>d_m (mm)</th><th>a/d_m</th><th>c (mm)</th><th>σ_{pc} (N/mm²)</th><th>A_s (mm²)</th><th>b_w (mm)</th><th>V_{exp} (kN)</th></tr>
<tr><td>I</td><td>TB1</td><td>23.22</td><td>40.2</td><td rowspan="3">650</td><td>214</td><td>450</td><td>1.44</td><td>550</td><td rowspan="5">0</td><td rowspan="7">760.2</td><td>150</td><td>149.0</td></tr>
<tr><td rowspan="2">II</td><td>RB2</td><td>0</td><td>38.8</td><td>240</td><td rowspan="2">240</td><td rowspan="2">2.71</td><td>-</td><td rowspan="6">200</td><td>68.8</td></tr>
<tr><td>TB2</td><td>10.39</td><td>34.4</td><td>130</td><td>600</td><td>75.8</td></tr>
<tr><td rowspan="4">III</td><td>RB3-0</td><td>0</td><td rowspan="4">50.5</td><td rowspan="4">1250</td><td>250</td><td rowspan="4">250</td><td rowspan="4">5.00</td><td>-</td><td>63.3</td></tr>
<tr><td>TB3-0</td><td>7.13</td><td>100</td><td>1200</td><td>56.2</td></tr>
<tr><td>RB3-3</td><td>0</td><td>250</td><td>-</td><td rowspan="2">3</td><td>95.5</td></tr>
<tr><td>TB3-3</td><td>7.13</td><td>100</td><td>1200</td><td>106.6</td></tr>
</table>

α_t: taper slope; f_c': compressive strength of concrete; a: shear span; d_s: effective depth at support; d_m: effective depth at mid-span; c: length of the tapered part; σ_{pc}: average prestress level at the cross section of loading point; A_s: cross section area of tensile reinforcement bars; b_w: width of the beam; V_{exp}: shear capacity from the experiments.

2-7-2 Longitudinal reinforcement (dowel action) effect

(Nghiep 2011) [63], he studied the longitudinal reinforcement effect in his research, the result is in (Fig. 2-11). (Briefly, main outcome of his research for this point was that tensile bars had high effect on shear capacity).

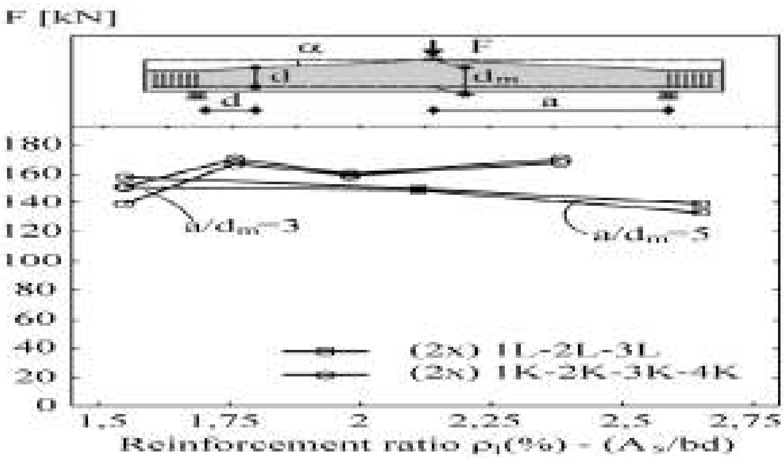


Fig. 2-11 Relation between reinforcement ratio & failure load [63].

(Albegmprli et al. 2018) [55], they are studied comprehensive experimental investigation on the mechanical behaviour for types of RCHBs by casted twenty RCHBs with four prismatic beams one of their parameters was influence of inclination flexural reinforcement RCHBs the authors were concluded that vertical component of tensile stress that occurs in the longitudinal reinforcement does cause positive effect on shear capacity, as in the Tables (2-7 and 2-8) and Fig. 2-12.

Table (2-7) Beams specification by [55].

Beam	Mode	α^{o} [1]	h_o [2] (mm)	h_s [3] (mm)	A_s [4] (mm^2)	$A_{s'}$ [5] (mm^2)
B0-0	-	0	300	300	603 (3Ø16)	100 (2Ø8)
B1-0	B	4.97	300	250	603 (3Ø16)	100 (2Ø8)
B2-0	B	9.87	300	200	603 (3Ø16)	100 (2Ø8)
B2-1	B	9.87	300	200	402 (2Ø16)	100 (2Ø8)
B3-0	B	14.62	300	150	603 (3Ø16)	100 (2Ø8)
B3-1	B	14.62	300	150	402 (2Ø16)	100 (2Ø8)

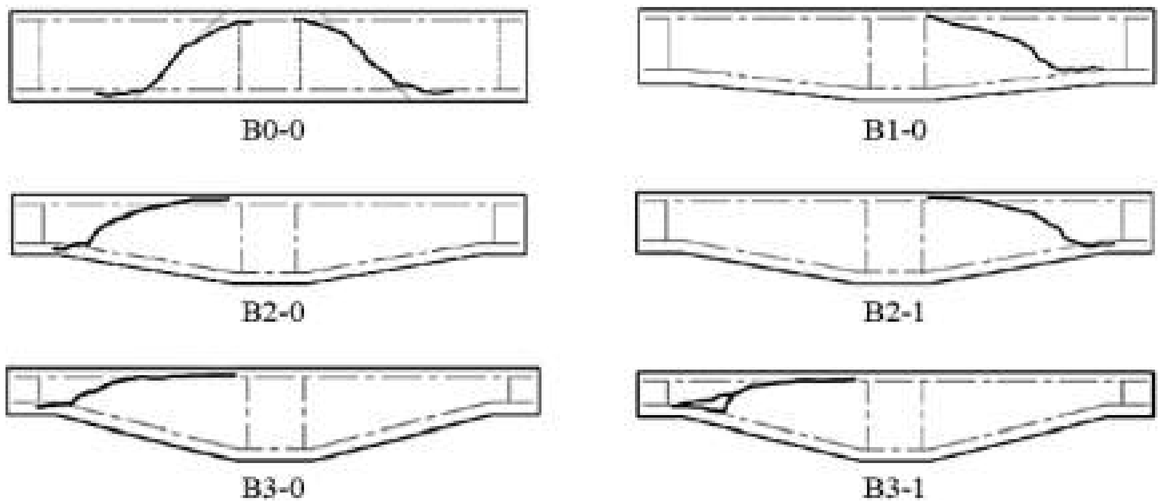


Fig. 2-12 Beams geometry [55].

Table (2-8) Test result of beams [55].

Beam code	f_c MPa	E_c GPa	f_t^1 MPa	Load kN First crack	Load kN Shear crack	Load kN Collapse	Peak Displacement mm	Effective depth mm	Failure Mode
B0-0	55	34.1	3.9	42	110	110.3	1.96	260	S
B1-0	53.5	35.2	4.65	39	108.2	108.2	1.9	250	S
B2-0	55.1	50.6	4.26	37	117	117	2.24	215	S
B2-1	53.9	33.2	4.0	36	91	91	2.57	215	S
B3-0	59.5	36.38	4.36	36	109	132	4.64	180	S
B3-1	51.5	35	3.6	33	111.2	123	4.57	180	S

2-7-3 Inclination angle effect (size effect)

(Kani 1967) [64], with 4-groups of tested beams with no transverse bars, and with various height of (6-in,12-in,24-in, and 48-in), with constant tensile bars ratio and strength of concrete, affirmed that, shear strengths decrease when beam's thickness increases. And at the same level, the tests those carried out by (Shioya et al. 1989) [65], on beams were had variable thickness (4in. to120in.) also he gave same inference. To clarify that phenomenon, (Collins et al. 1986) [66], and (Reineck, 1991) [11], they are supposed that crack width at failure is proportional to the beam's depth. Since wide crack width will reduce shear transfer capability due to aggregate interlock and friction, the higher the beam depth the lower the shear stress transfer capacity.

(Nghiep, 2011) [63], he studied inclination angle effect in his research that mentioned in Effect of a/d, the result is in (Fig. 2-13). (Briefly, main outcome was that inclination had high effect on shear capacity).

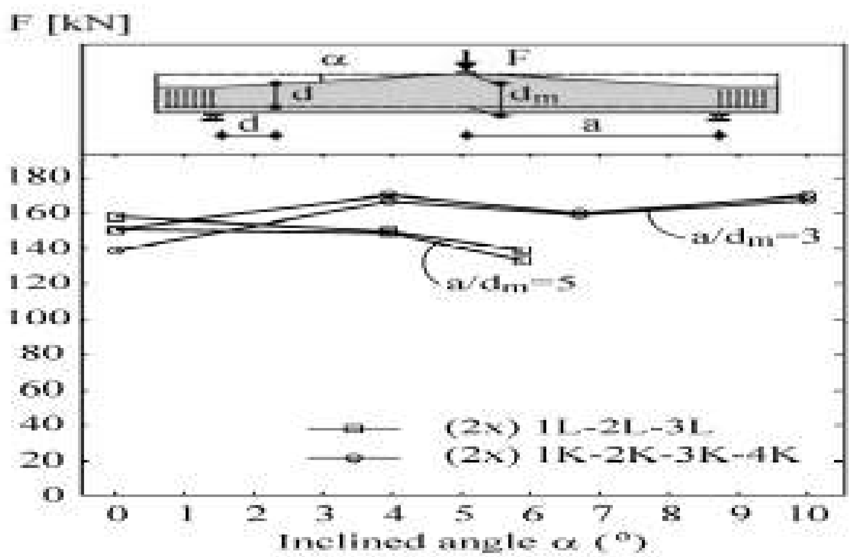


Fig. 2-13 Relation between inclination angle & failure load [63].

(Albegmprli et al. 2018) [55], they were studied comprehensive experimental investigation on the mechanical behaviour for types of RCHBs by casted twenty RCHBs with four prismatic beams one of their parameters was influence of inclination angle, the authors have concluded that stress's vertical component that occurs in the reinforcement does contribute to the load capacity of them and this means increases the capacity.

2-8 Shear capacity and CFRP bars/strips

Banding plate of steel to tension zone of concrete's member by adhesive resin was workable technique to increment shear strength, and flexural strength of beam. Several bridges, and buildings were strengthening by FRP technique; because the steel plate could corrode, and caused retro gradation of their bond to concrete substrate; and due to their installation's onerousness, which requires utilize of heavy equipment, researchers have started utilizing FRP materials as alternate to steel plate, external post tensioning, and section enlargement e.g. concrete column jacketing (ACI 440.2R-08) [67]. The using of FRP composites for strengthening structure was firstly studied at Swiss Federal Laboratory for Material Testing and Research (EMPA) by Prof. U. Meier, and his team in mid of 1890s (Motavalli et al. 2016) [68]. Since; considerable research studies in shear, and flexural strengthening of structure had carried out, mostly in USA, Europe, and Japan (ACI 440.2R-08) [67].

Table (2-9) represent comparison between steel reinforcement and CFRP types (ACI 440.1R-06) [69].

Table (2-9) Comparison between steel reinforcement and FRP types [69].

	Steel	**GFRP**	**CFRP**	**AFRP**
Nominal yield stress, ksi (MPa)	40 to 75 (276 to 517)	N/A	N/A	N/A
Tensile strength, ksi (MPa)	70 to 100 (483 to 690)	70 to 230 (483 to 1600)	87 to 535 (600 to 3690)	250 to 368 (1720 to 2540)
Elastic modulus, ksi (GPa)	29.0 (200.0)	5.1 to 7.4 (35.0 to 51.0)	15.9 to 84.0 (120.0 to580.0)	6.0 to 18.2 (41.0 to 125.0)
Yield strain, %	0.14 to 0.25	N/A	N/A	N/A
Rupture strain, %	6.0 to 12.0	1.2 to 3.1	0.5 to 1.7	1.9 to 4.4

FRP consisting of high strengths fiber embedded in polymer resin. Fiber is major constitutive that carry's loads, and has wide range of stiffness and strength. Carbon CFRP, Aramid AFRP, and Glass GFRP fibers are the common reinforcement utilized with FRP composites. The comparison among CFRP, AFRP, GFRP and steel bars in term of stress strain relation are clarify in (Fig. 2-14) (Alnatit 2011) [70].

Some FRP advantages are as pursue (ISIS 2007) [71]:

- Lightweight;
- High strength;
- Corrosion resistance;
- Durable;
- High longitudinal resistance;
- Easy to install;
- Impregnable to electromagnetic environment;
- Reducing maintenance cost, and;
- Increasing infrastructure service life.

And some FRP disadvantages are as pursue (ISIS 2007) [71]:

- Fibers are ruptured without yielding;
- Abrupt degeneration of characteristics under high temperature;
- Low modulus of elasticity, and;
- Low transverse strength.

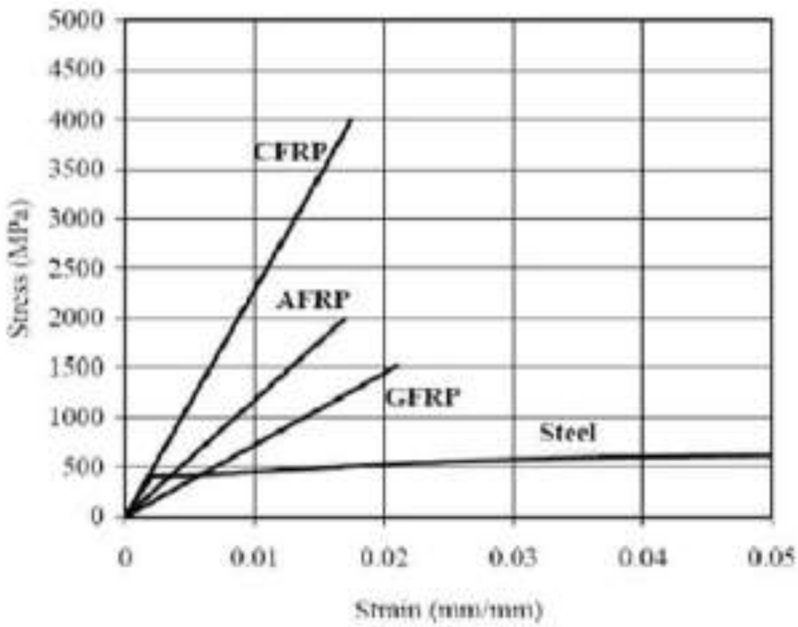


Fig. 2-14 Comparison among CFRP, AFRP, GFRP, and steel bars in term of stress strain relationship [71].

RC beam's shear failure mode must be averted; because it's unpredictable, and brittle.

CFRP material is being utilized as a competitive alternative on RC structures rehabilitation. For shear strengthening; there're two main techniques for CFRP's applied as the pursues;

2-8-1 Externally bonded reinforcement EBR strips/sheets

Widespread shear strengthening configuration of this technicality comprise:

i. Fully wrapping.
ii. Side bonding; and
iii. U-wrapping.

(Fig. 2-15 and Fig. 2-16), (ACI 440.2R-08) [67].

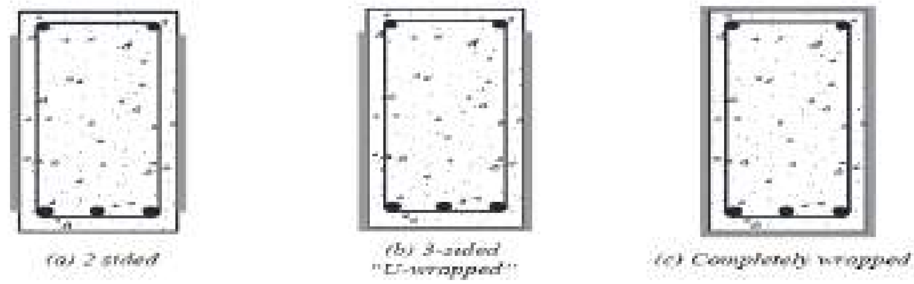


Fig. 2-15 Typical wrapping schemes for shear strengthening using FRP laminates according to [67].

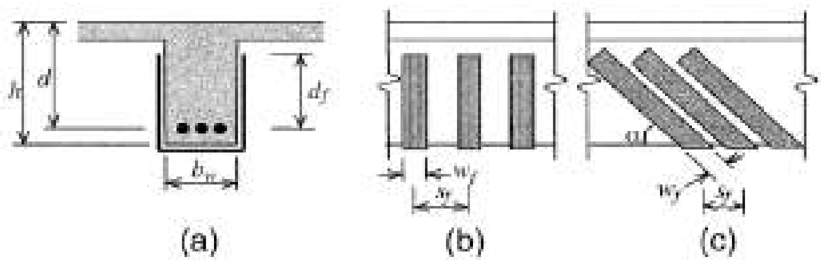


Fig. 2-16 Illustration of dimensional variables used in shear strengthening calculations for repair, retrofit, or strengthening using FRP laminates according to [67].

For external FRP reinforcement in form of discrete strips, center to center spacing between strips shouldn't surpass totally of d/4 plus strip's width.

2-8-2 Near surface (NS)

It's ones of the more favorable strengthening techniques for structures of concrete. Research on this topic started only a few years ago, but has by now attracted worldwide attention. The method eliminates many of the surface preparation issues, critical to successful implementation, and efficacy, associated with field layup externally bonded CFRP systems. Since the bar is bonded to the member on three sides, development length is much shorter and it is possible to utilize the full strength of the bar. Unlike field layup FRP system, there is no need for highly skilled and trained FRP installation expert. Design is dictated by (ACI 440.2R) [67].

This technique including makes slots on beam's surface to insert laminates/rods inside those slots, FRP rods/strips bonded to concrete by utilizing a convenient epoxy adhesive (Fig. 2-17). In typically, CFRP laminate has cross sectional about 1.4mm thick, and (5 mm - 50mm width), while FRP rod could has (6mm- 8mm- 12mm) diameter.

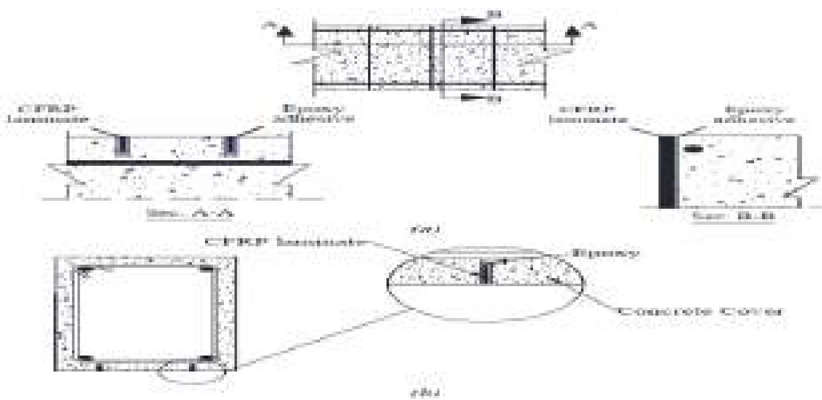


Fig. 2-17 NS FRP to super fat the beam's (a) shear capacity, (b) bending capacity [67].

(Lorenzis & Nanni 2002) [72], the objective of their research was to study NSM FRP rods' bonding with concrete. There were twenty-two inverted T-beams in their experimental programme, and their factors were FRP material type CFRP & GFRP, bonded length, slot's size in which rod embedded, diameter and the rod's configuration sandblasted & deformed surface. The test was done by utilized four point loads with shear span [19 in. (483 mm)]. Their concluded were the deformed rod is more effective than sandblasted rod in terms of bond performance, when failure occurred due to the epoxy's cover splitting, increasing the size of slot led to higher bond strength, increasing slot's size, and thickness of cover, leads to a higher bond strength when the failure ruled by the splitting of cover's epoxy. Ultimate load increases correspondingly, and failure may finally occur by surrounding concrete instead of epoxy. Ultimate load increased, as predicted, with rod's bonded length. Table (2-10).

Table 2-10 Results of tested beams [72].

Specimen code	Type of FRP rod/rod size	Surface configuration	Bonded length (No. of d_b)	Groove size, in.	Ultimate pullout load, lb	Percentage of ultimate tensile load of rod, %	Average bond strength, psi	Failure mode*
G4D6a	GFRP/ No. 4	Deformed	6	5/8	5548	24	1177	SOE
G4D12a			12	5/8	7778	34	825	SOE
G4D12b				3/4	8307	36	881	SOE + C
G4D12c				1	9628	42	1022	SOE + C
G4D18a			18	5/8	9563	41	676	SOE
G4D24c			24	1	13,918	60	738	SOE + C
C3D6a	CFRP/ No. 3	Deformed	6	1/2	3523	18	1329	SOE
C3D12a			12	1/2	6006	20	1133	SOE
C3D12b				3/4	6880	23	1298	SOE + C
C3D12c				1	6472	22	1221	C
C3D18a			18	1/2	9452	32	1189	SOE
C3D24b			24	3/4	9880	33	932	SOE + C
C3S6a	CFRP/ No. 3	Sandblasted	6	1/2	2965	12	1119	SOE
C3S12a			12	1/2	3927	16	741	PO
C3S12b				3/4	3460	14	653	PO
C3S12c				1	3931	16	742	PO
C3S18a			18	1/2	5602	23	704	PO + SOE
C3S24a			24	1/2	5025	20	474	PO + SOE
C4S6	CFRP/ No. 4	Sandblasted	6	5/8	5082	13	1078	SOE
C4S12			12		5839	14	620	PO + SOE
C4S18			18		6634	16	469	PO + SOE
C4S24			24		7934	20	421	PO + SOE

*SOE = splitting of epoxy; C = concrete cracking; and PO = pullout.
Note: 1 in. = 25.4 mm; 1 lb = 4.448 N; and 1 psi = 6.89 kPa.

(Dias and Barros 2010) [73], the effectiveness of NSM technique with CFRP laminates for shear strengthening of T-beams was their aim. They were casted fifteen T-beams, with three inclinations of laminates were 45˚, 60˚, and 90˚, with three different stirrups and FRP ratios, they were deduced that NSM more efficient from EBR in order to NSM was provided greater increasing in load capacity after shear cracks formulation. NSM doesn't demands surface preparation work, and after cutting and cleaning thin slots strengthening procedure is resumed to CFRP laminates installation minimal time requires for installation, a further advantage for this technique that when NSM is utilized, the outside semblance of structural member not affected by intervention of strengthening.

(Dias & Barros 2017) [74], effectiveness of NSM with CFRP laminates for shear strengthening of T-beams was their aim. Their experimental programme are comprised nine T-beams five of them were as reference beams and other with NS CFRP shear strengthening T-beams, their variables were, stirrups' number in shear span zone (a) and number & CFRP's inclination angle. All beams are tested by three point loads. (Fig. 2-18) shows (Dias & Barros) variables and testing load type. They're deduced that NSM shear strengthening with CFRP laminate is highly efficient in beams to assuring higher mobilization of CFRP's tensile capacity.

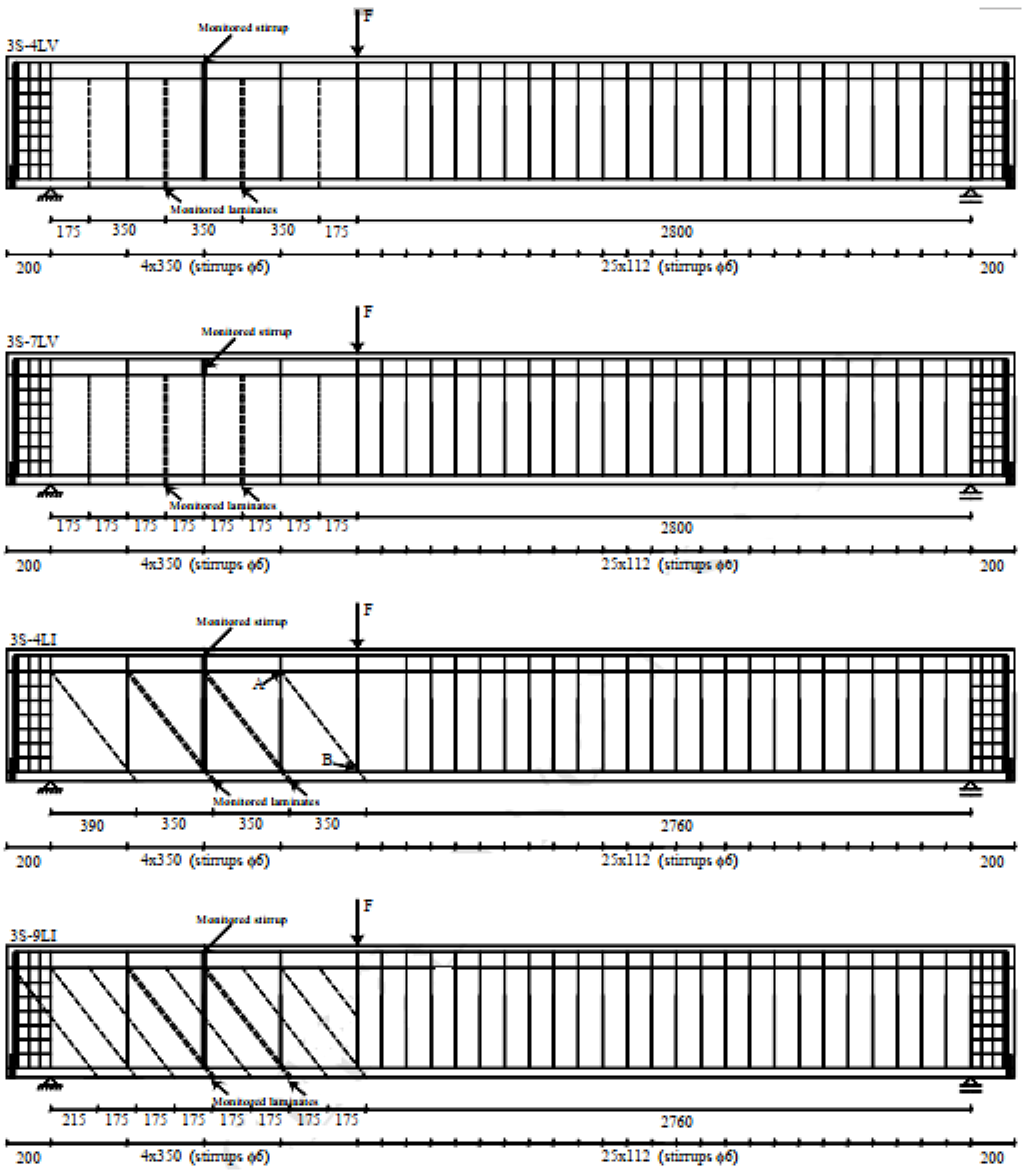


Fig. 2-18 Dias & Barros variables and type of testing load [74].

2-8-3 Contribution of CFRP in shear capacity

The FRP sheet/strip shear capacity count on numerous factors, e.g., FRP's modulus of elasticity, FRP's thickness that applied on concrete's surface, FRP's orientation, concrete's compressive strength, and FRP's application technique. Predicting shear strength contribution and understanding shear failure mechanisms of FRP had been the study's object for many researchers like (Zhang 2005) [75]. Some existent models are presented in following:

2-8-3-1 CFRP strip's contribution in shear capacity

- ***ACI 440 Model***

The FRP shear strength participation were proposed in (ACI 440) [67] is based on research offered by (Khalifa et al. 1998) [76] and (Khalifa et al. 2000) [77] as following:

$$\Phi V_n = \Phi\ (V_c + V_s + \psi\ V_f) \qquad \text{..... (29)}$$

$$V_f = [A_{fv} * f_{fe} * (\sin\alpha + \cos\alpha * d_{fv}]/\ s_f \qquad \text{..... (30)}$$

$$A_{fv} = 2\ n_{tf} * w_f \qquad \text{..... (31)}$$

$$f_{fe} = \varepsilon_{fe} * E_f \qquad \text{..... (32)}$$

(n_{tf}) are layers' number of (FRP sheet/strip) applied. $\Phi = 0.75$, (A_{fv}) FRP's Area, (d_{fv}) FRP's Effective depth, (α) FRP's inclination angle, (w_f) FRP width, (s_f) Space c/c sheet, (f_{fe}) stress in FRP, (ε_{fe}) FRP strain, (E_f) modulus of elasticity of FRP.

$$\varepsilon_{fe} = kv * \varepsilon_{fu} \leq 0.004 \quad \text{for U-jacking and two sides bounding} \qquad \text{....(33)}$$

$$\varepsilon_{fu} = C\ \varepsilon_{fu} \qquad \text{....(34)}$$

$$k_v = [(k_1 * k_2 * Le)/(11900 * \varepsilon_{fu})] \leq 0.75 \qquad \text{....(35)}$$

$Le= [(23300/ (n_f * t_f *E_f)^{(0.58)}]$ .... (36)

$k_1= (f_c'/27)^{(2/3)}$ .... (37)

$k_2 = (d_{fv} -L_e)/ d_{fv}$ for U-jacketing bounding .... (38)

$k_2 = (d_{fv} – 2* L_e)/ d_{fv}$ for two sides bounding ....(39)

$(V_s + V_f) \leq (0.66\sqrt{f_c'} * b_w d)$ shear strength limits ....(40)

- ***Triantafillou's Model***

The efficiency of load carrying capacity of externally bonded FRP shear reinforcement at ultimate limit state count on FRP failure's mode, i.e., FRP's tensile fracture or FRP de-bonding. The FRP tensile fracture could occurs at stress lower than FRP tensile strength owing to stress concentrations at de-bonded areas or rounded corners. Nevertheless, it's hard to know which failure's mode will occur, since failure's mode count on series of varied factors, such as available anchorage length, bonding conditions, FRP's modulus of elasticity and concrete, FRP's thickness. In practice, actual mechanism is combination of both tensile fracture and peeling of FRP. Based on mentioned considerations, (Triantafillou ACI 1998) [78] and (Triantafillou et al. 2000) [79] had proposed approach to find shear strength contribution for FRP externally bonded, in which shear strength participation due to FRP given by:

$V_{frp}= 0.9\, d\, b_w\, \rho_{frp}\, E_{frp}\, \varepsilon_{frp}\, (1+\cot \alpha) \sin \alpha$ ....(41)

$\rho_{frp}= 2\, t_{frp}\, w_{frp} / (b_w\, s_{frp})$ ....(42)

$\varepsilon_{frp} = \min. [0.65\, ((f_c')^{(2/3)}/(\rho_{frp}\, E_{frp}))^{(0.56)} * 10^{-3}$,

$0.17((f_c')^{(2/3)}/(\rho_{frp}\, E_{frp})^{(0.3)}\, \varepsilon_{frp}]$ for U-jacketing and (two-sides) bonding of carbon FRP]. .....(43 & 44)

Where;

(V_{frp}); FRP's contribution to shear capacity, (ρ_{frp}); is FRP reinforcement ratio, (***α***) steel inclination angle.

2-8-3-2 NS CFRP bar's Contribution in Shear capacity

(De Lorenzis & Nanni) [80] are proposed two models to predicted NS FRP bar contribution in shear capacity for beams, with reference to failure mechanisms, the first V_{1F} is FRP shear strength participation concerning to bonding shear failures, second V_{2F} is FRP shear strength participation corresponding to maximum FRP strain, as follows:

$$V_{1F} = 2\pi\ d_b\ \tau_b\ L_{tot} \qquad ....(45)$$

$$\tau_b = 0.001\ (d_b\ E_b)\ /\ L_i) \qquad ....(46)$$

$$L_{tot} = d_{net} - S \;\Longrightarrow\; if \quad (d_{net}/3) \leq S \leq d_{net} \qquad ....(47)$$

$$L_{tot} = 2\ d_{net} - 4\ S \;\Longrightarrow\; if \quad (d_{net}/4) \leq S \leq d_{net}/3 \qquad ....(48)$$

$$d_{net} = d_r - 2\ c \qquad ....(49)$$

Where; d_b is bar's diameter, τ_b is average bond strength, L_{tot} is sum of effective lengths of whole bars those crossed by crack, E_b is modulus of elasticity of bar, L_i is effective length of rod that crossed by crack corresponding to tensile strain, d_{net} is reduced length of FRP rod, S is spacing of NS CFRP bar, d_r is height of shear strengthened part of cross sectional, c is concrete's cover.

$V_{2F} = 2p\ d_b\ t_b\ L_i \quad (d_{net}\ /\ 3) \leq S \leq (d_{net}/2)$, *$V_{2F}$ controls if $L_i > S$. If $L_i < S$* ....(50)

V_{2F} controls with the value.

$V_{2F}=2\pi\ d_b\ \tau_b\ (L_i + d_{net} - 2S)$ ....(51)

$If\ (d_{net}\ -2S) < L_i < S$ ....(52)

$V_{2F}= 4\pi\ d_b\ \tau_b\ L_i \quad if \quad L_i < d_{net} - 2\ S$ ....(53)

$(d_{net}\ /\ 4) < S < (d_{net}\ /\ 3)\ V_{1F}$ *controls if* $L_i > d_{net} - 2\ S$ ....(54)

if $L_i < d_{net} - 2\ S$, V_{2F} *controls with the value*

$V_{2F}= 2\pi\ d_b\ \tau_b\ (L_i + d_{net} - 2S) \quad if$ ....(55)

$S < L_i < d_{net} - 2\ S$

$V_{2F}= 2\pi\ d_b\ \tau_b\ (2L_i + d_{net} - 3S) \quad if$ ....(56)

$d_{net} - 2\ S < Li < S$

$V_{2F}= 6\pi\ d_b\ \tau_b\ L_i \qquad if$ ....(57)

$L_i \leq d_{net}\ -3S$

Preliminary approaches presented above includes two formulas those may be utilized to find VFRP, and suggests by authors taking the lower value from the two formulas as the NSM FRP rods participation to shear capacity, they also concluded that utilize of NS FRP bar is effective technique to promote the shear capacity of RC beams.

The present research deals with the following parameters: Inclination angle, number of opening and its position, shear span to effective depth, longitudinal reinforcement ratio, number of stirrups, near surface CFRP bar orientation, CFRP strip orientation, and steel fiber ratio. The tapered-beam will be UHPC, and this type of beam with such concrete type doesn’t investigated before. So this research will find out the effect of each parameter mentioned above on such beam with discussion three method of designing and will recommend which method is suitable to design tapered- beam casted with UHPC mixture.

CHAPTER THREE

EXPERIMENTAL WORK

3-1 GENERAL

This Chapter deals with experimental study of the shear strength for the UHPC tapered-beams have longitudinal holes to investigate the effect of these holes on the structural element in the term of shear capacity, crack strength, crack pattern, deflection, and mode of failure. Those holes had been made by utilized PVC pipes to utilization it for passing of service runs, and to reduce the self-weight of the concrete tapered-beams. The first objective of this experimental work was to acquire UHPC properties, thirty-nine concrete mixtures, twenty-five of them illustrative in Table (3-1) were made with different attribution to reach the target UHPC mixture, and the maximum acquired compressive strength was 170MPa which is UHPC. The second experimental programme deals with casting of nineteen UHPC tapered-beams. The specimens had been casted, and then tested, the test was done under the effect of two point loads to find the influence of presence of hollows in the UHPC tapered-beam section, in the terms of: variable number of hollows, Inclination angle, longitudinal reinforcement, shear span to effective depth a/d ratio, with or without shear reinforcement (stirrups), shear strength by utilizing NS CFRP bars and CFRP strips in three orientations (30°, 45°, and 0°), and steel fiber ratio. The molds, details of specimens, the concrete mixes, and testing are dealt in this chapter.

Table (3-1) Proportion of mixtures.

Mixture	Cement kg/m^3	Steel fiber by		PC 260 Cement + Silica fume		Sand #2 kg/m^3	Sand #3 kg/m^3	Sand #4 kg/m^3	Sand #5 kg/m^3	Silica fume ratio by		W/C ratio by	
		%	kg/m^3	%	kg/m^3					% from cement	kg/m^3	% from cement	kg/m^3
1	1167	2	157	2.25	32.8	0	0	667	0	25	292	25	292

2	1167	2	157	2.25	32.8	0	0	667	0	25	292	23	268.4
3	1000	2	157	2.75	35.7	0	1000	0	0	30	300	19	190
4	1000	2	157	2.75	35.7	0	1000	0	0	30	300	22	220
5	1000	2	157	2.75	35.7	0	1000	0	0	30	300	18.5	185
6	1000	2	157	2.75	34.4	0	1000	0	0	25	250	17.5	175
7	1000	2	157	2.75	34.4	0	1000	0	0	25	250	21	210
8	1000	2	157	2.75	34.4	0	1000	0	0	25	250	19	190
9	800	2	157	2.75	28.6	0	1000	0	0	30	240	19	152
10	800	2	157	3	28.6	0	0	1000	0	30	240	21	168
11	1000	2	157	3	39	0	0	1000	0	30	300	21	210
12	1029	2	157	3	40.1	0	910	0	0	30	309	20	206
13	1029	2	157	3	40.1	0	0	910	0	30	309	23	236.6
14	1029	2	157	3	40.1	0	0	910	0	30	309	21	216
15	1029	2	157	3	40.1	910	0	0	0	30	309	18	185.2
16	1020	2	157	3	38.3	0	900	0	61	25	255	23	234.6
17	1020	2	157	3	38.3	0	0	1020	211	25	255	27.5	280.5
18	1000	2	157	3	37.5	0	600	340	60	25	250	24.3	243
19	1000	2	157	3	33	0	1000	0	0	10	100	18	180
20	1000	2	157	3	37.5	0	0	1000	0	25	250	21	210
21	1000	2	157	3	33	1000	0	0	0	10	100	18	180
22	1000	2	157	3	39	1000	0	0	0	30	300	18	180
23	1000	2	157	2.75	34.4	0	0	1000	0	25	250	23	230
24	1029	2	157	3	40.1	0	0	910	0	30	309	24	247
25	1000	2	157	3	40.2	958	0	0	0	34	340	20	200

3-2 The task schedule in present work

The working was divided in to many stages, the first was to get the target UHPC mixture after that the structure's models UHPC tapered-beams were casted. The graph in the (Fig. 3-1) clarifies by simplified way the work strides:

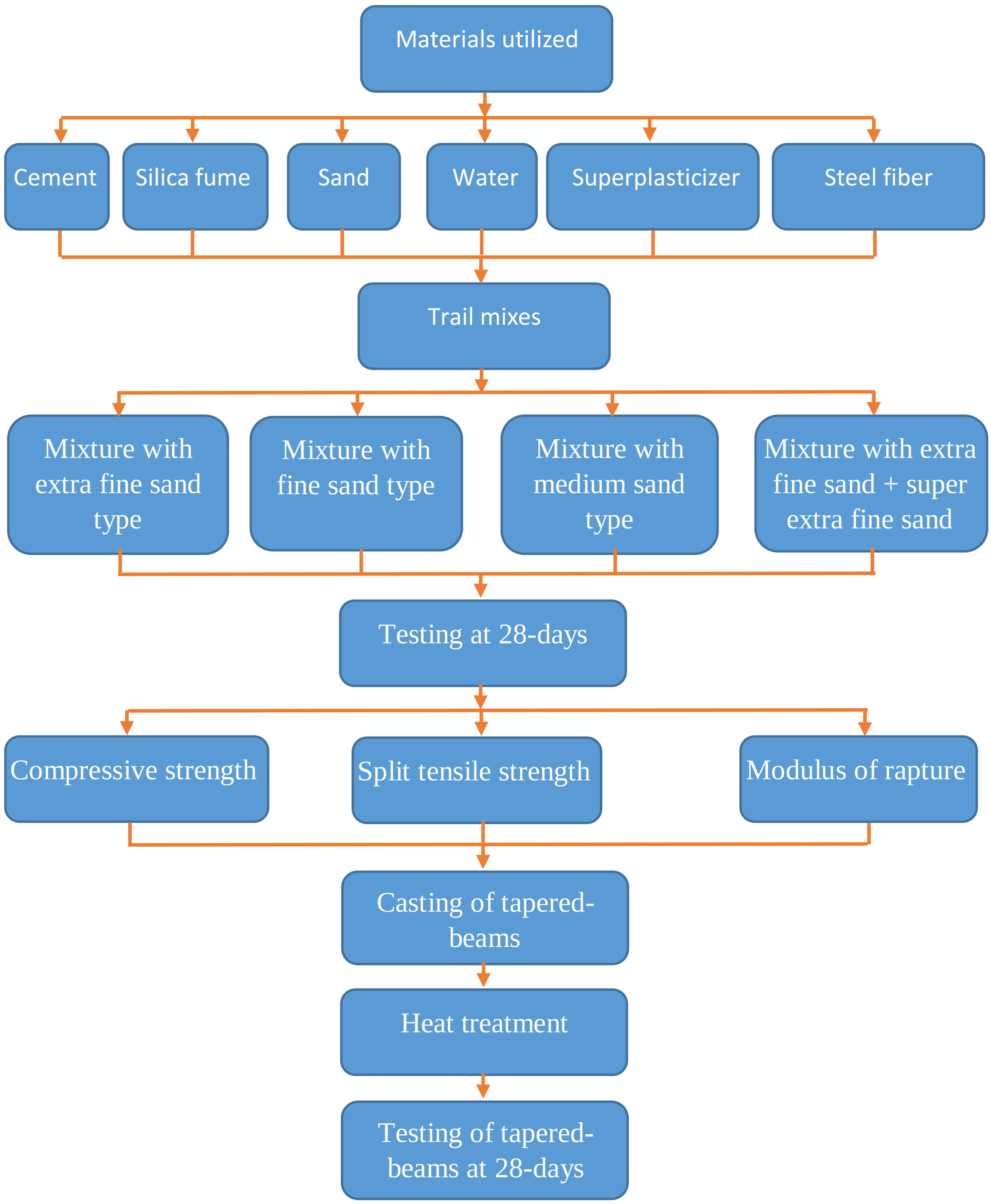


Fig. 3-1 Strides of the present work.

3-3 UHPC's mix design

Till now, there is no codes or international standards deal with UHPC mixture, but there are many researches and previous studies had been done for the mix design and obtained UHPC strength under laboratory's conditions, because UHPC needs special mixer and conditions this including the casting process, and curing procedure.

3-3-1 Materials

The target of UHPC mixture is not easy to get because; of several reasons, the UHPC mix material components are too expensive, UHPC special mixer isn’t available in the local market, UHPC is new and uncommon or famous to utilize, in addition to what have been mentioned, the laboratory is too poor and hasn’t the simplest requirements to product UHPC mixture, and the same applies to laboratory's heat-treatment conditions, so the research is compelled to manufacture a water tank for heat-treatment, with maximum heat 80°C and it is in the range of AFGC [29] which states that raising the components temperature (90°C ± 10°C), and bought special mixer with 20L capacity for UHPC trial mixtures.

3-3-1-1 Cement

The type of the cement that utilized in all the mixtures of this study was ordinary Portland cement elucidate the cement's type. The physical and chemical properties of cement were also examined in Amarah Technical Institute laboratory, the results are elucidating in Tables (3-2 and 3-3), where the results elucidate that this type is in conformity with the (ASTM C150) [81] for physical and chemical properties, and the Iraqis specification standard IQS No.5 for chemical properties. The amount of cement is utilized as a variable in this research.

Table (3-2) Physical properties of cement.

(Physical properties)	Test Result	(ASTM C150)
(Smoothness)	(0.2%)	(5 %) Maximum
Setting-time utilizing Vicat's instrument Initial (min.) Final (hr.)	 (90) (4.0)	 (45) Minimum (10) Maximum
Compressive strength at: (3days MPa) (7days MPa)	 (15.2) (19.5)	 (12) Minimum (19) Minimum
[Fineness utilizing Blain-air permeability apparatus (m^2/kg)]	(380)	(280) Minimum

Table (3-3) Chemical properties of cement.

(No.)	Compound composition	Chemical composition	Content %	IQS %	ASTM C150 % max.
1-	Lime	(CaO)	60.4	-----	-----
2-	Alumina	(Al_2O_3)	5.32	-----	6
3-	Iron oxide	(Fe_2O_3)	5.5	-----	6
4-	Sulphate	(SO_3)	2.5	2.8	3
5-	Loss on ignition	(L.O.I)	1.6	4	3
6-	Lime saturation	(L.S.F)	0.69	0.66 -1.022	
7-	Tricalcium	(C_3A)	7.6	-----	8
8-	Silkone oxide	(SIO_2)	27.21	-----	-----

3-3-1-2 Silica fume

In recent years, there have been a clear interesting in utilizing pozzolanic silica as an improved material for concrete properties as additional ratio for cement this what is made in this research, or as a replacement ratio for a partly of cement. Silica that was utilized in this study is micro-silica, or as known silica fume, this kind is widely available in the local markets in the form of sacks, weighing 20kg (Fig. 3-2) and Table (3-4) are elucidated the results of silica fume test. The ratio of silica fume is utilized as a variable in this research.

Fig. 3-2 Micro-silica (silica fume).

Table (3-4) Result of tested silica fume.

(Color)	(Grey to medium grey powder)
(Specific Gravity)	(2.1 to 2.4)
(Bulk Density)	(500 to 700 kg/m)
(Chemical Requirement)	
[Silicon dioxide $(SiO)_2$]	(Minimum 85%)
[Moisture Content (H O)]	(Maximum 3%)
[Loss on ignition (LOI)]	(Maximum 6%)
(Physical requirements)	
(Specific Surface-area)	(Minimum 15 m/g)
(Pozzolanic activity-index, 7days)	(Maximum 105% of control)
[Over-size particles retained on (45) micron-sieve]	(Maximum 10%)

3-3-1-3 Fine aggregate (sand)

The fine aggregate (sand) that utilized for UHPC production must be within certain sizes differ from ordinary sand. Four gradations of sand were utilized for UHPC development, sand#2 has medium gradation 1-2mm, sand#3 has fine gradation 0.75 mm - 1.5mm, sand#4 has extra fine gradation 0.3 mm - 0.6mm, and sand#5 has super extra fine gradation 0.08mm - 0.15mm, those were produced by Don Construction Products Ltd. DCP (Appendix B), and available in the local markets in the form of sacks weighing 25kg. The sand's four types were elucidating in (Fig. 3-3). The type of sand is utilized as a variable in this research.

Fig. 3-3 Utilized sand's types.

3-3-1-4 Water

The drinking water produced by Reverse Osmosis method (RO) is utilized in the pour of whole of the trial mixtures and casting of concrete-tapered beams; because the tap water isn't suitable for concrete purposes because it has high percentage of salt, the water was brought to the laboratory by water tanker from the nearest (RO) water station. The w/c ratio is utilized as a variable in this research.

3-3-1-5 High Range Water Reducing Admixture (HRWRA) Superplasticizer

As it's known, the lower water content, the greater strength gets, this means the reducing the water percentage in the concrete mixture plays a big role in obtaining UHPC, therefore, another material should be provided to compensate for this large decrease in the water ratio.

In a simplified way the addition of superplasticizer to concrete mixture allows the reduction of w/c ratio without negatively effecting on mixture workability. Here comes the role of the superplasticizer for the purpose of giving the concrete workability on the one hand and increase the concrete strength on the other hand.

The superplasticizer PC260, produced by (DCP) (Appendix B) utilized was complies with (ASTM C494) type (A&G) [82] (Fig. 3-4), and its technical description is elucidated in Table (3-5). The superplasticizer is utilized as a variable in this research.

Fig. 3-4 Superplasticizer PC260.

Table (3-5) Technical properties of PC260.

Technical properties @ 25˚ C	
Color	Yellowish to brownish liquid
Freezing point	≈ -7˚ C
Specific gravity	1.1 ± 0.02
Air entrainment	Typically less than 2% additional air is entrained above control mix at normal dosages.

3-3-1-6 Steel fibers

Steel fiber that was utilized is golden and straight type (Fig. 3-5), manufactured by China, it's available in local markets in form of sacks weighing (20 - 25kg). Steel fibers that utilized have 0.2 mm diameter, and 13 mm length with aspect ratio 65, the content of steel fiber was 2% by volume. Table (3-6) is elucidated the characteristics of steel fiber.

Fig. 3-5 Type of steel fiber.

Table (3-6) Steel fiber's characteristics.

Product code	Dia.	Len.	L/D ratio	Ten. Strength	Type
Copper plated steel fiber	0.2 mm	13 mm	65	2800 MPa	Loose

Mixture proportions are summarizing in Table (3-1). The content of cement, water, silica fume, and sand were the parameters in UHPC mixtures' development. Silica fume in this research was utilized in different percentages an additional not replacement quantity of cement to study silica fume's effects on concrete's compressive strength.

In addition to cylindrical specimens, and cube specimens, (0.1m * 0.1m * 0.1m), were casted to evaluate UHPC 19 cubes compressive strength. The cubes treatment and testing procedure was comparable to that of cylinders' specimens (Graybeal 2007) (104-M17) [83]. Water tank was manufactured and utilized for heat curing.

3-3-1-7 Steel reinforcement

The steel bar that utilized in the all specimens was 2Φ8mm in the compression zone to forming of reinforcement cage. the steel bar that used in the tension zone reinforcement was 2Φ25mm, except four concrete tapered-beams two of them belong to third group the first had 4Φ16mm were distributed in two rows, and the second had 2Φ16 + 2Φ12mm were distributed in two rows, the other two tapered-beam are belong to eighth group had 4Φ16mm distributed in two rows. The tapered-beams from group one to group seven were without shear reinforcement. The group eight had variable shear reinforcement (stirrups) without, 4 stirrups, and 5 stirrups Φ 8 mm. All bars reinforcement those utilized were Ukraine brand. Three samples for each diameter were tested to find the yield stress (f_y) and ultimate stress (f_u). The tests were carried out at the laboratory technical institute of Amarah. Table (3-7) elucidated results of tested bars (results are the average of three bars for each size). Results were in accordance with ASTM (A615/A615-15) [84].

Table (3-7) Test result of steel reinforcement.

Test Results				ASTM A615/A615M-04B limits		
Bar size (mm)	Yield strength (N/mm²)	Ultimate strength (N/mm²)	Elongation (%)	Yield strength Min.(N/mm²)	Ultimate strength Min.(N/mm²)	Elongation (%)
8	423.49	526.4	28.8	427.3	529.35	28.9
12	577.6	653	11.6	350	550	7
16	551.6	655.7	10.2	350	550	7
25	481.3	671.6	13.3	350	550	7

3-4 Design of tapered-beams

3-4-1 Design for flexural strength

Three methods were utilized for design of flexural strength, Irregular section method, Deep beam method, and Nasser's formulas for (reactive powder concrete) [24] that based on the JSCE [45]. The three methods were organized in excel sheet, appendix-A-.

3-4-1-1 Irregular section method

This design method is based on the trial and error process, in this process, a lever arm from the center of gravity of the compression block to the center of gravity of the steel is estimated to equal the larger of (0.9 d), and from this value, called z, a trial steel area is calculated. Then by the process utilized in appendix-A-, the value of the estimated lever arm is checked. If there is much difference, the estimated value of z is revised and a new A_s determined. This process is continued until the change in A_s is quite small and in the same time the determined z value is too close to the estimated value. All design of tapered-beams was organized by excel sheet, appendix-A-.

3-4-1-2 Deep beam method

The member called deep beam if it satisfies one of the following conditions (clear span (l_n) ≤ 4 h, and shear span (a) ≤ 2 h). The design method for compressive and tensile reinforcement is based on the strut and tie model. The compressive strut should roughly follow the direction of the compressive stress trajectories as shown by the refined and simple strut and tie model in the (Fig. 1-4) that applied when two point loads effect. The results were organized in excel sheet, appendix-A-.

3-4-1-3 Nasser's method [24]

The equations those mentioned the chapter two are utilized in the calculation of the main longitudinal reinforcement, with deduct the openings from each compressive and tensile areas, and utilized the force in the tensile steel bar instead the force in the FRP bar. The method was organized in excel sheet, appendix-A-.

3-4-2 Design of shear reinforcement

To design the tapered-beams for shear strength with/without stirrups, the deep beam method and Nasser's formulas [24] were adopted, with utilizing (Albegmprli et al. 2018) formula [55], that mentioned in the chapter two to calculate dowel action contribution in shear capacity. To calculate the contributions of CFRP strips in shear strengthening are utilized (ACI 440) [69] in addition to Nasser's formulas [24] with Albegmprli formula [55]. To calculate the contributions of NS CFRP bars in shear strengthening are utilized (Lorenzis & Nanni) formulas [80] in addition to Nasser's formulas and Albegmprli formula were utilized, it's illustrative in appendix-A-.

3-5 The aim of study

The main aim of this study is to assess the effect of longitudinal opening on the shear behaviour of UHPC tapered-beams with NS CFRP bars. The subsequent parameters those affected shear capacity of tapered-beam were studied in current study which are:

1- Shear span to effective depth ratio a/d;

2- Tensile bar ratio;

3- Existence of steel stirrups, to know its contributions on the overall shear capacity;

4- Effect of inclination angle on shear capacity of tapered-beams;

5- Effect of number and location of openings on shear capacity of tapered-beams;

6- The effect of CFRP bars with different orientation 30°, 45°, and 0° on the shear capacity of tapered-beams;

7- The effect of CFRP strips with different orientation 30°, 45°, and 0° on the shear capacity of tapered-beams and make a comparing between CFRP bars and CFRP strips.

8- Effect of steel fiber ratio (0%, 1%, and 2%) on shear capacity of tapered-beam, and comparing between steel fiber ratio and stirrups;

9- Make a comparing between CFRP bars and stirrups;

10- Make a comparing between CFRP strips and stirrups.

All tapered-beams were without shear reinforcement or with 8mm as steel stirrups, the steel reinforcement of UHPC tapered-beams and NS CFRP deformed bars (Fig. 3-6) and CFRP strips 50 mm width (Fig. 3-7)for UHPC tapered-beams were designed to guarantee shear failure occurring. The maximum load of flexural failure and shear failure were chosen, Appendix (A).

Fig. 3-6 NS CFRP bar (deformed type).

Fig. 3-7 CFRP strip.

3-6 Specimens and parameters

The study consists of nineteen UHPC tapered-beams including twelfth groups, the total length of each tapered-beam is 1.9 m with clear span 1.6m, the width is 0.15m, depth at ends (prismatic zone) H1 is 0.18m, the depth at mid span (tapered zone) H2 was 0.405m (2.25 H1), except two concrete tapered-beams included in first group had (H2) 0.315m (1.75H1), and 0.36m (2 H1) respectively, all the openings were circular made by utilized PVC pipes with diameter Φ50 mm. Table (3-8) is elucidated the detail of all groups. Each tapered-beam is denoted by TB which means tapered-beam.

Table (3-8) Detail of groups.

No. of group	**Beam ID**	**Tensile bar**	**Stirrups or CFRP**	**a/d**	**No. of openings**	**Inclination angle**	**Steel fiber%**
First group	TB 2	2Φ25mm	without	2.73	Two	9.7°	2
	TB 3					12.8°	
	TB 10					15.9°	
Second group	TB 5	2Φ25 mm	Without		One in H1	15.9°	2
	TB 6				One in H2		
	TB 9			2.3	Two		
Third group	TB 7	(2Φ12+2Φ16)mm	Without	2.73	Two	15.9°	2
	TB 8	4Φ16 mm					
	TB 10	2Φ25mm					
Fourth group	TB 1	2Φ25 mm	Without	2.94	Two	15.9°	2
	TB 9			2.3			
	TB 10			2.73			
Fifth group	TB 10	2Φ25 mm	Without	2.73	Two	15.9°	2
	TB 11		4 NS CFRP bars				
	TB 12						
	TB 13						
	TB 10	2Φ25mm	Without	2.73	Two	15.9°	2
	TB 14						

Sixth group	TB 15		4 CFRP strips				
	TB 16						
Seventh group	TB 10	2Φ25mm	Without	2.73	Two	15.9°	2
	TB 17		5Φ8mm				
	TB 18		4Φ8mm				
Eighth group	TB 8	4Φ16mm	Without	2.73	Two	15.9°	2
	TB 19						0
	TB 20						1
Ninth group	TB 11	2Φ25mm	4 CFRP strips	2.73	Two	15.9°	2
	TB 12						
	TB 13						
	TB 14	2Φ25mm	4 NS CFRP bars				
	TB 15						
	TB 16						
Tenth group	TB 11	2Φ25mm	4 NS CFRP bars	2.73	Two	15.9°	2
	TB 12						
	TB 13						
	TB 10		Without				
	TB 17		5Φ8mm				
	TB 18		4Φ8mm				
Eleventh group	TB 14	2Φ25mm	4 CFRP strips	2.73	Two	15.9°	2
	TB 15						
	TB 16						
	TB 10		Without				
	TB 17		5Φ8mm				
	TB 18		4Φ8mm				
Twelfth group	TB 8	4Φ16mm	Without	2.73	Two	15.9°	2
	TB 19						0
	TB 20						1
	TB 10	2Φ25mm	Without				2
	TB 17		5Φ8mm				
	TB 18		4Φ8mm				

3-6-1 First group

This group was including three UHPC tapered-beams, TB 2, TB 3, and TB 10, with two longitudinal openings, and variable inclination angle 9.697°, 12.835°, and 15.897° with total depth at mid span 0.315m, 0.36m, and 0.405m, respectively. The purpose of this group was to study the effect of the inclination angle on the shear capacity of section, details of the group in the (Fig. 3-8).

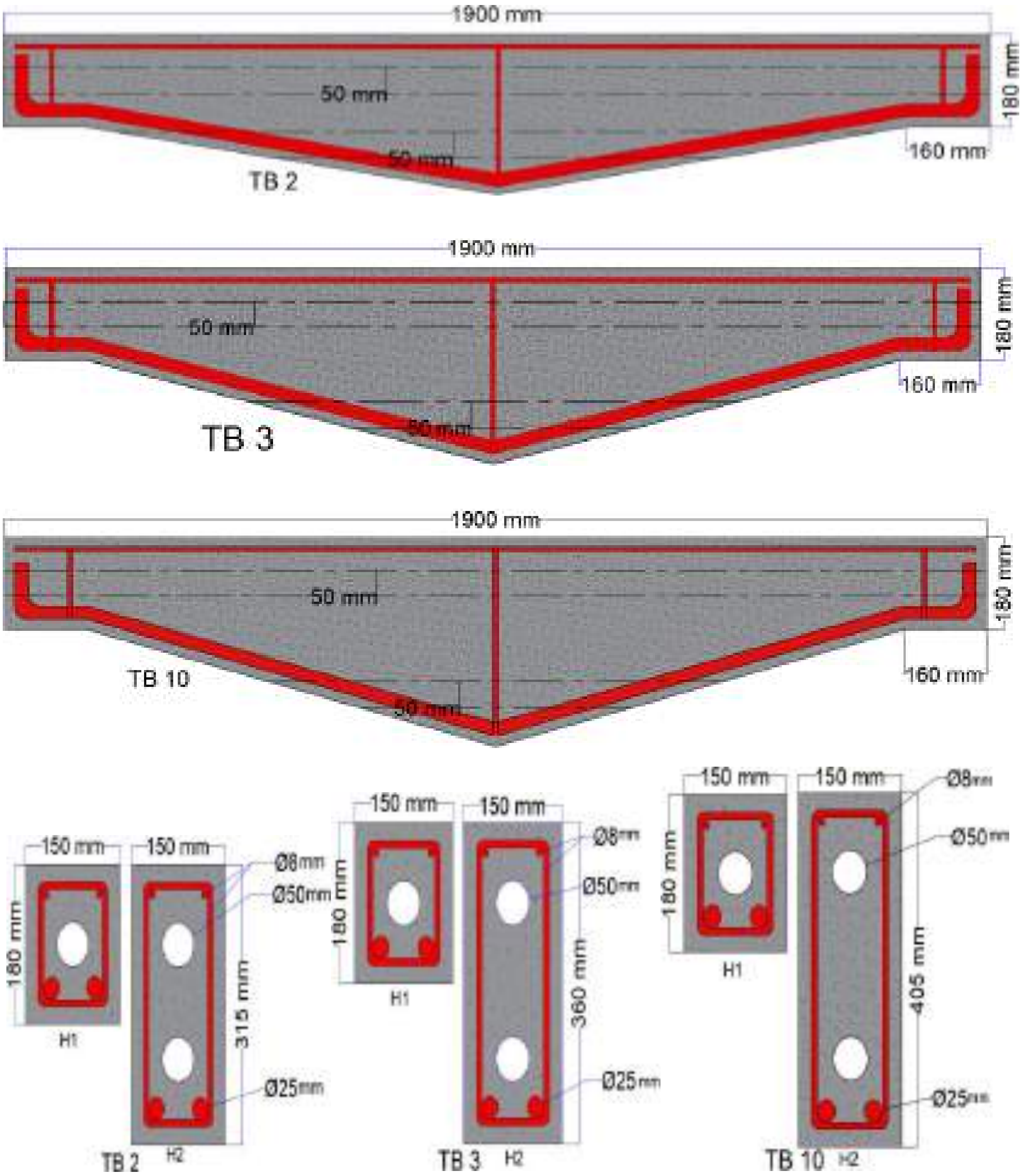


Fig. 3-8 First group details.

3-6-2 Second group

This group was containing three UHPC tapered-beams the first one was TB 9 with two openings in the prismatic and tapered zone, the second with one opening in the prismatic zone TB 5, the third was with one opening in the tapered zone TB 6, the a/d of group is =2.3, this group's aim was to deduct the effect of number and location of opening on the shear capacity of tapered-beams, details of the group in the (Fig. 3-9).

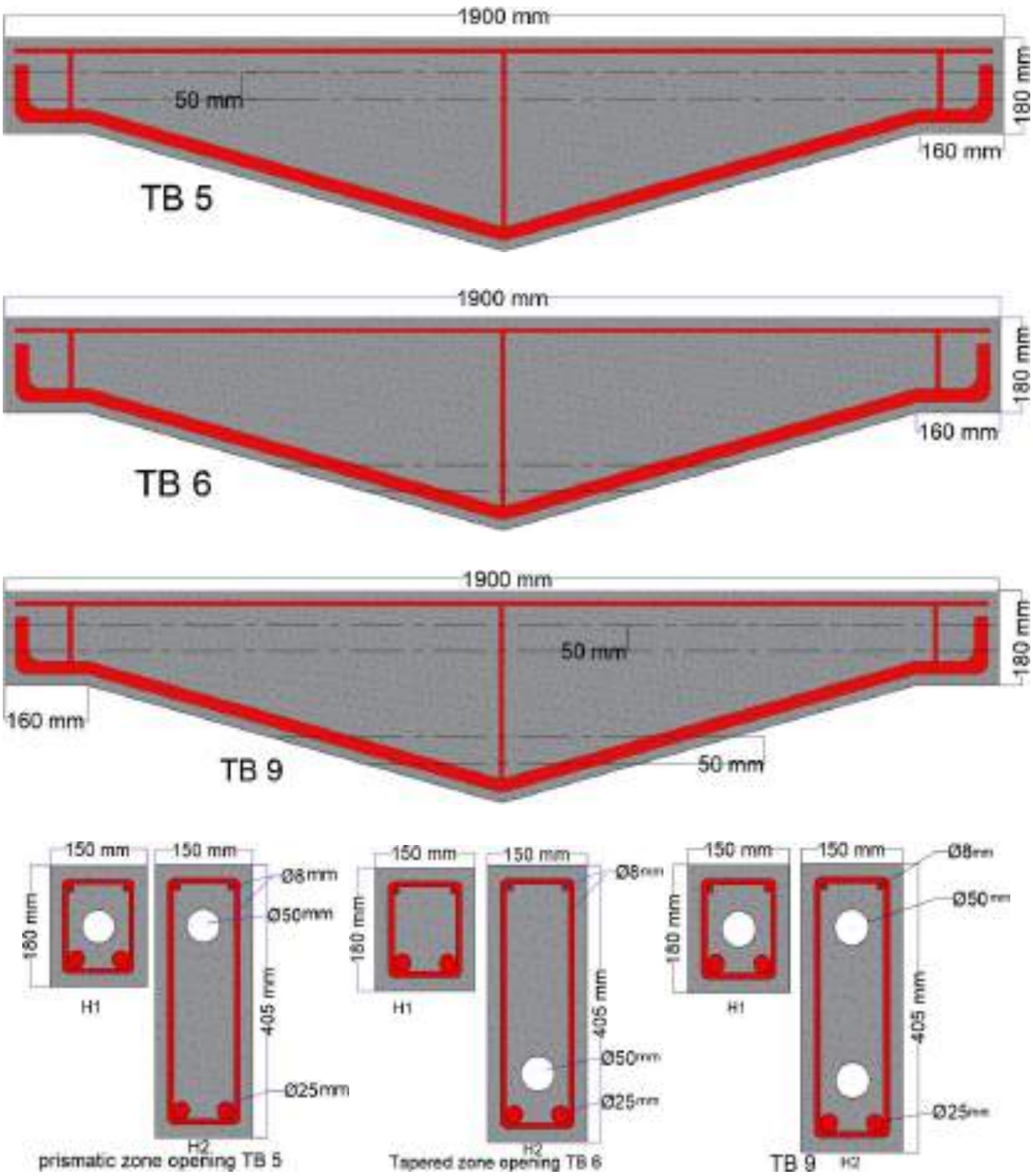


Fig. 3-9 Second group details.

3-6-3 Third group

This group was inclusive three UHPC tapered-beams, the first one was from group one TB 10 and new two concrete tapered-beams TB 7 and TB 8. All of them had two longitudinal openings, with different longitudinal reinforcement in the tension zone (2Φ25, 2Φ16+ 2Φ12mm, and 4Φ16) respectively, the objective of this group was to find out the effect of the longitudinal reinforcement (dowel action) on the shear capacity of concrete tapered-beams, details of the group in the (Fig. 3-10).

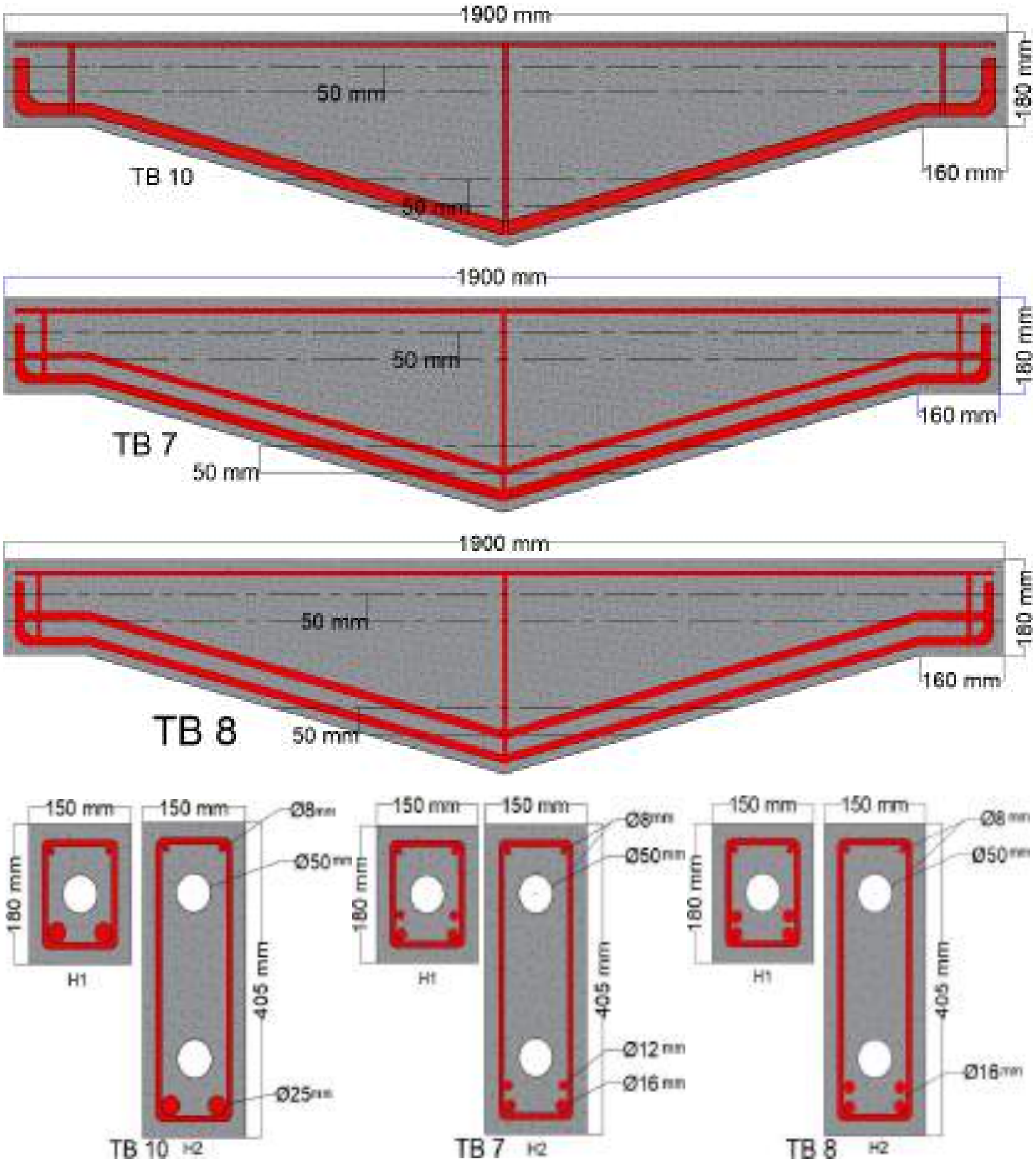


Fig. 3-10 Third group details.

3-6-4 Fourth group

Three UHPC tapered-beams contained in this group, the first was from first group TB 10 and the others were two concrete tapered-beams TB 1 and TB 9. All of them had two longitudinal openings, and had the same tensile bars in the tension zone 2Φ 25mm with a/d ratio (2.73, 2.94, and 2.3) respectively, the aim of this group was to study the effect of the shear span to effective depth a/d ratio on shear capacity of concrete tapered-beams, details of the group in the (Fig. 3-11).

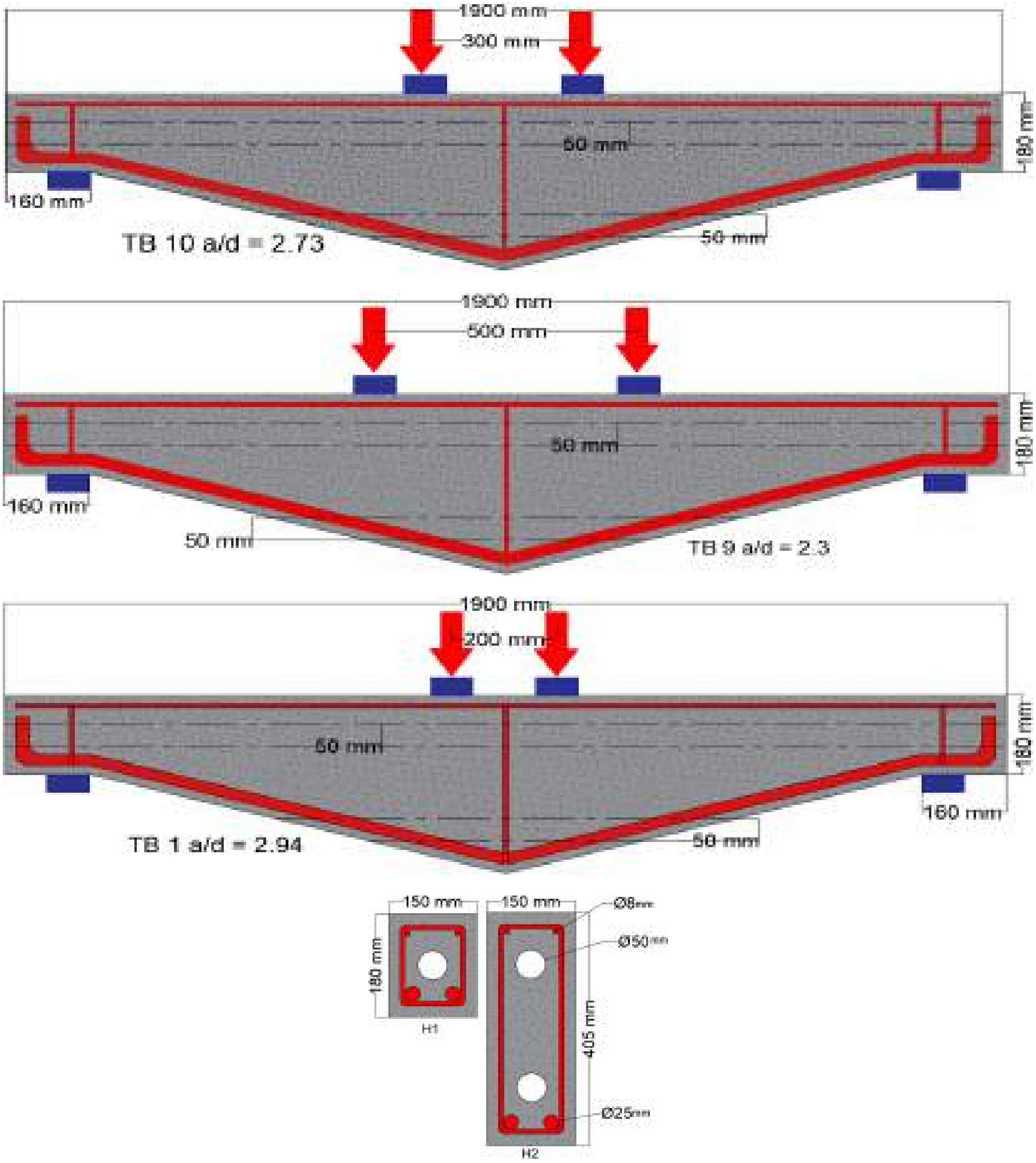


Fig. 3-11 Fourth group details.

3-6-5 Fifth group

Four UHPC tapered-beams belong to this group, the first was from first group TB 10 without NS CFRP bars and the others were three concrete tapered-beams TB 11, TB 12, and TB 13. All of them had two longitudinal openings, and had the same longitudinal reinforcement in tension zone 2Φ 25mm, the aim of this group was to study NS CFRP bars strengthening effect with different orientations (without, 0°,45°, and 30°) on the shear capacity of tapered-beams, group's details in the (Figs. 3-10, and 3-12).

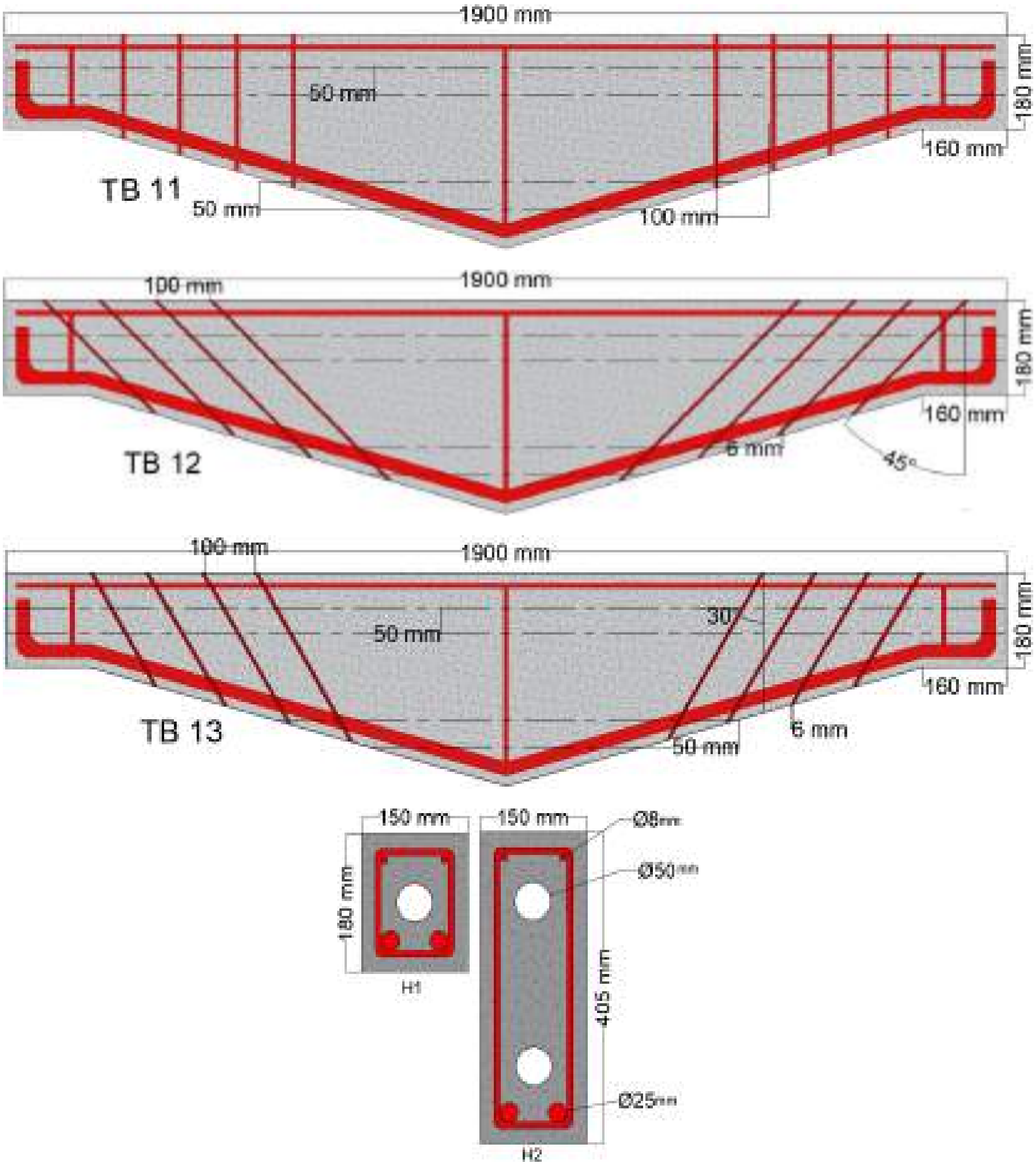


Fig. 3-12 Fifth group details.

3-6-6 Sixth group

In this group there were four UHPC tapered-beams, one from first group TB 10 without CFRP strips, and the others were three concrete tapered-beams TB 14, TB 15, and TB 16. All of them had two longitudinal openings, and had same longitudinal reinforcement in tension zone 2Φ 25mm. The aim of this group was to study CFRP strips (50mm width were executed by U-wrapped) strengthening effect with different orientations (without, 0°,45°, and 30°) on shear capacity of concrete tapered-beams, details of the group in the (Figs. 3-10 and 3-13).

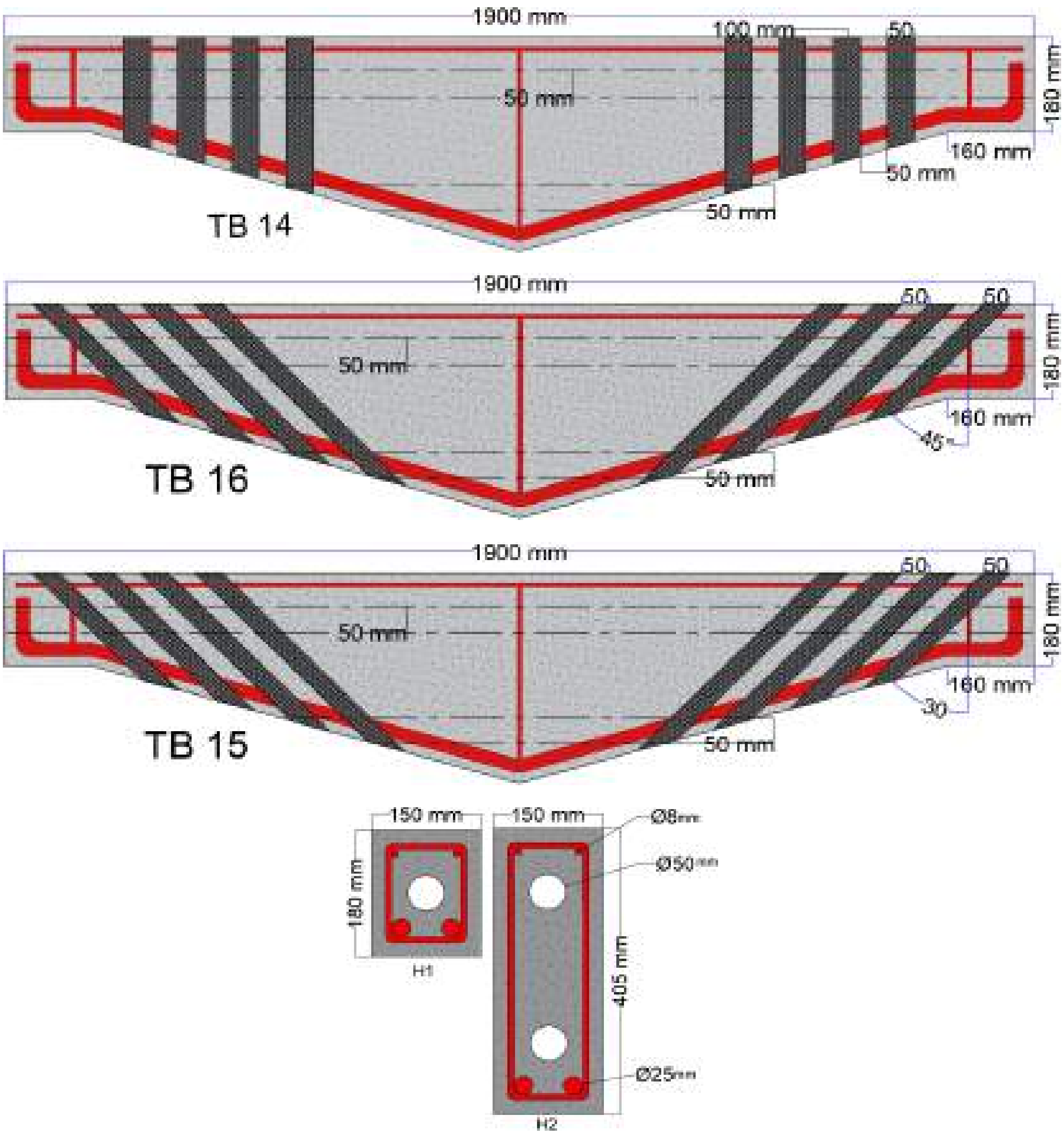


Fig. 3-13 Sixth group details.

3-6-7 Seventh group

In this group there were three UHPC tapered-beams, one was from first group TB 10 without stirrups, the second TB 18 was with four shear reinforcement, and the third TB 17 was with five shear reinforcement (stirrups). All of them had two longitudinal openings, and had the same longitudinal reinforcement in the tension zone 2Φ 25mm the aim of this group was to investigate the shear reinforcement on the shear capacity of tapered-beam, details of the group in the (Fig. 3-14).

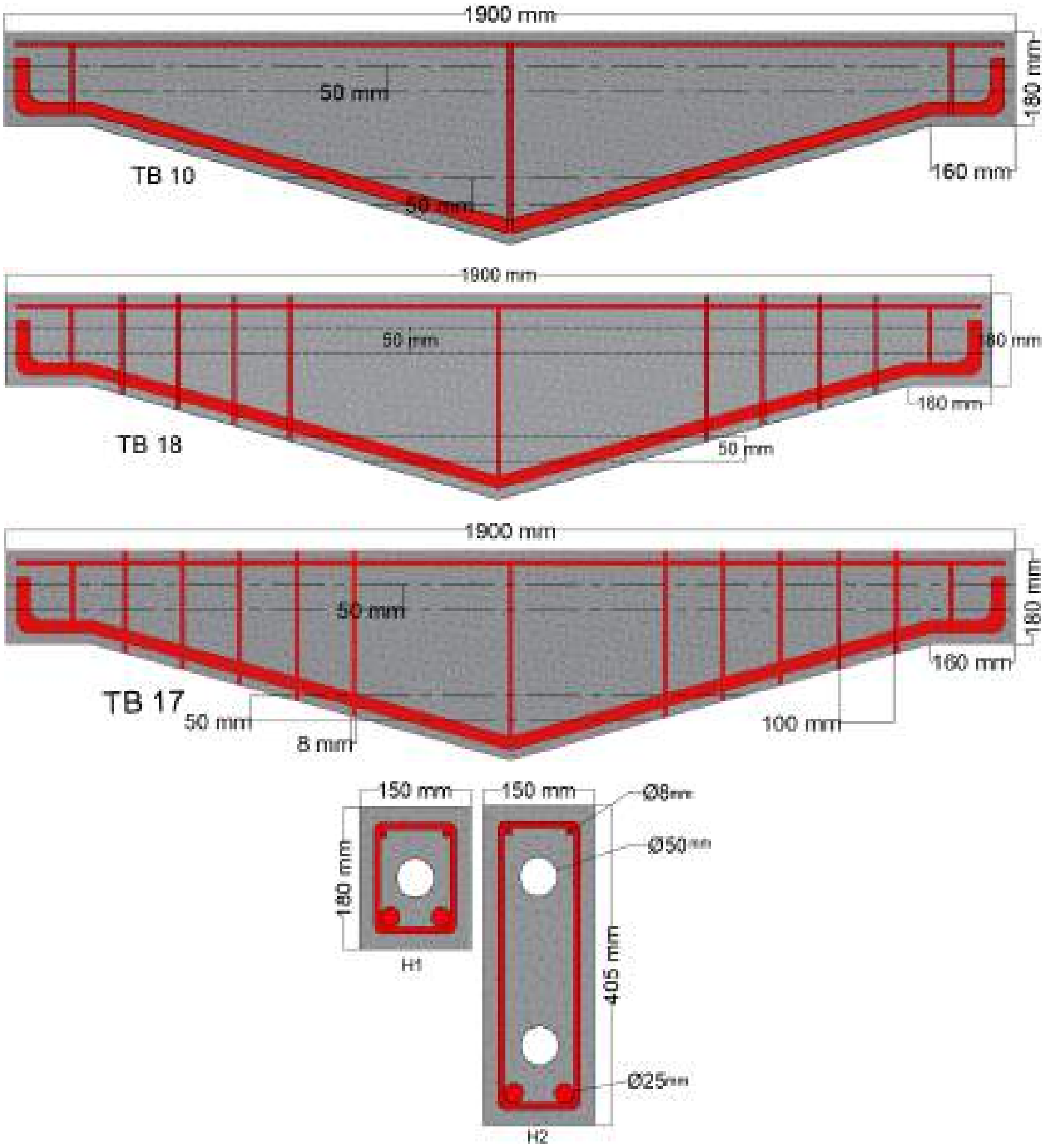


Fig. 3-14 Seventh group details.

3-6-8 Eighth group

This group is included three UHPC tapered-beams, one from third group TB 8, and the other were two tapered-beams TB 19, and TB 20. All of them had two longitudinal openings, and had same longitudinal reinforcement in tension zone 4Φ16mm, with variable steel fiber ratio 2%, 0%, and 1% respectively, the purpose of this group was to study the effect of steel fibers ratio on section's shear capacity, details of the group are same third group details 4Φ16mm.

3-6-9 Ninth group

In this group there were seven UHPC tapered-beams, the first is tapered-beam TB 10, and the tapered-beams of group five TB 11, TB 12, and TB 13, to compare with tapered-beams of sixth group TB 14, TB 15, and TB 16. The aim of this group was to compare between NS CFRP bars and CFRP strips with respect to with respect to w.r.t shear capacity of tapered-beam to find out which strengthening technique is better.

3-6-10 Tenth group

This group was inclusive six UHPC tapered-beams, the first is tapered-beam TB 10, and the tapered-beams of group five TB 11, TB 12, and TB 13, to compare with tapered-beams from seventh group TB 17 and TB 18. The objective of this group was to make a comparing between NS CFRP bars and stirrups w.r.t shear capacity of tapered-beam to find out the contribution of NS CFRP bars and stirrups on increasing of shear capacity.

3-6-11 Eleventh group

This group was inclusive six UHPC tapered-beams, the first is tapered-beam TB 10, and the tapered-beams of group six TB 14, TB 15, and TB 16, to compare with the tapered-beams from seventh group TB 17 and TB 18. The objective of this group was to make a comparing between CFRP strips and stirrups w.r.t shear capacity of tapered-beam to find out the contribution of CFRP strips and stirrups on increasing of shear capacity.

3-6-12 Twelfth group

This group was inclusive six UHPC tapered-beams, the first is tapered-beam TB 10, and the tapered-beams of group seven TB 17, and TB 18, to compare with the eighth group TB 8, TB 19, and TB 20. The objective of this group was to make a comparing between stirrups and steel fiber w.r.t shear capacity of tapered-beam to find out the contribution of steel fiber and stirrups on increasing of shear capacity.

3-7 Fabrication (Molds)

Nineteen of UHPC tapered-beams molds were fabricated from plywood blocks 18mm thickness, from three sides to get smooth surfaces each two tapered-beams were together in one mold (Fig. 3-15). All the longitudinal hollows were made by utilizing PVC pipes with diameter Φ50mm with 1.95m length, which means longer than the length of tapered-beam by 5cm, 2.5cm from each side to ensure that PVC pipe is installed in its place, with filling the gaps between PVC pipe and the plywood mold by pistol silicon glue. Openings in plywood were made by driller.

Fig. 3-15 Fabrication of plywood molds.

3-8 Concrete casting

In this study the UHPC trial mixes were mixed by utilizing 18 L pan-mixer that manufactured in the local market according to the requirements. The processes of UHPC trial mixture were in strides as summarized below:

1. Cement and silica fume (cementitious materials), were mixed for (1.5min.) with slow motion of mixer;

2. Sand was added slowly over cementitious, with continue mixing the dry materials with slow motion of mixer for another (1.5min.);

3.Water and PC260 were mixed together, and added half of resulted liquid to admixture slowly and continue mixing for (3min.) with increase the speed of mixer to medium motion;

4. Half of remaining liquid was added slowly to admixture, and continue mixing for another (3min.);

5. Then the rest liquid was added to admixture, the mixing time of this process isn't specified but will vary for all mixtures due to the low w/c ratio, and high content of binder. During this process the style of the admixture will change progressively, from a dry to a dry with balls, and finally to be a thick paste. At this process the speed of mixer was maximum motion, and;

6. The steel fibers were added slowly (to prevent forming of steel fiber balls) to mixture about one minute. Continued mixing for three minutes to mix steel fibers well with other components as in (Fig. 3-16).

Fig. 3-16 Adding of steel fiber.

It's clearly from the stages those mentioned above that UHPC needs high speed mixer with extended mixing time.

The surfaces of all specimens were covered by wet sand layer (sand type#4).

The same strides mentioned above were utilized in tapered-beams' casting by utilized (mixture No. 24 that mentioned in (Table 3-1), for two reasons; it's workability and there were enough quantity of sand#4 in the laboratory), but by using bigger pan mixer that also manufactured in local market with capacity 40L.

It is recommended that concrete be poured without interruption. In the case of a discontinuous process with interruptions of concreting, or in the case of a long delay between two batches, a skin may form on the surface of the last concrete layer poured. Surface desiccation must be avoided and concrete layers must be joined together by raking the interface surface for example to ensure fiber continuity [29].

3-9 Casting of specimens

Nineteen of tapered-beams specimens were casted inside tanks, three water tanks were utilized, two mixtures for each tapered-beam, the first one was casted half of tapered-beam from edge to edge of mold (Fig. 3-17), and the second mixture to complete casting of tapered-beam, with utilized an electrical vibrator to secure concrete compaction and preventing of cavitation. Nine standard cubes (0.1m × 0.1m × 0.1m), nine standard cylinders (0.1m × 0.2 m), three standard prisms (0.1 m × 0.1m × 0.5m) were casted from the same mixture. All the faces of molds, cylinders, cubs, and prisms were clean, and treated by utilize oil before concrete casting, and the surfaces of all specimens were covered by wet sand layer (sand type#4), (Fig. 3-18).

Fig 3-17 Casting of tapered-beam.

Fig. 3-18 Using of wet sand to cover specimens.

3-10 Curing of specimens

After 24 hours of specimens' casting the water tanks were filled with RO water. The molds of cubes, prisms, and cylinders were removed, and the samples were placed in the water tank. Each water tank had one heater available in the local market with maximum temperature 80°C. The period of heat treatment lasted for one week and the specimens stayed in the water at laboratory temperature to complete 28-days of age (Fig. 3-19). After 28-days all tapered-beams were move out from the tanks by utilized manual crane, and cleaned by water jet pump, to preparing the surfaces of specimens and paint by white color to detect the crack pattern (Fig. 3-20).

Fig. 3-19 Curing of specimens.

Fig. 3-20 Painting the specimens by white color.

3-11 Component epoxy impregnation resin

The type of resin that is utilized with NS CFRP bars and strips was Sikadur330: is two components (two packages the weight of the big one A, white paste Fig. 3-21) is four times the small one (B, grey paste Fig. 3-22), thixotropic epoxy that based impregnating resin, and adhesive. Its easy mix by utilize two trowels for at least three minutes, and also easy application by same trowels and impregnation roller or brush. Components (A+B) mixed will give light grey paste (Fig. 3-23), with mixing ratio (4:1) by weight (this elucidates the difference in the weight of the two packages). The curing time is 7-days according to the product recommendations. (Appendix B).

Fig. 3-21 Component A of Sikadur-330.

Fig. 3-22 Component B of Sikadur-330.

Fig. 3-23 Light gray paste of Sikadur-330 components (A+B) mixing.

3-12 Carbon fiber reinforced polymer CFRP

The carbon fiber materials are one technique that improving the performance by increases the ultimate capacity and improved the other characteristics of RC beams. So, the research will investigate shear behavior, and the shear behavior performance of UHPC tapered RC beams strengthened by CFRP bars and strips, and casted with or without openings.

3-12-1 CFRP bars Near Surface NS technique

NS FRP method is usually utilized in the existent RC elements to increase their load carrying capacity, that's possible by adding tension FRP bars in the surface grooves those were made along the cover of tension side for flexural strengthening, or in the web for shear strengthening. NS technique is in particular attractive for flexural strengthening in the negative moment regions for slabs, decks, beams, and where the concrete shows section's damaged and requiring to cover's protective. NS becomes mostly interesting to be utilized in concrete elements' rehabilitation for old buildings, that makes NS technique has more attracted increasing amount of researches, and has been considerably applied. This technique was done according to the below stages. All the stages were executed according to (ACI440 2002) and followed to CFRP bars manufacturer recommendations:

(1) Diamond cutter was used to create grooves in concrete cover of tapered-beam with 1.67 d, (d = CFRP bar diameter) width and depth for NS CFRP bars (Fig. 3-24).

Fig. 3-24 Electrical diamond cutter to create grooves.

(2) The grooves were clean by utilizing water jetting machine.

(3) Mixing the two parts of epoxy materials (A and B) together.

(4) Filling full way (ACI 440.2R-08) [67]. Better than half way of slots with epoxy adhesive.

(5) CFRP bars were Insert into slots, with lightly pressed, this force led paste to out flow around CFRP bar and completely fill between groove's sides and bar. Then surface was leveled, and removed flowed epoxy adhesive by trowel. Fig. 3-25.

Fig. 3-25 Steps of NS CFRP bar installation.

3-12-2 CFRP strips technique

Subsequent strides are utilized in implementations of this kind of technique:

- The concrete surface was scraped by electrical abrasive paper; the paper's grade was (P 36);

- Tapered-beam's edges were rounded by electrical abrasive paper in order to prevent stress concentration at CFRP regions at corners of beam;

- Concrete surface were clean by water jet pump;

- CFRP strips and concrete surface were saturated with a convenient epoxy adhesive. (Fig. 3-26), and;

Fig. 3-26 Saturated of CFRP strips & concrete surface with epoxy.

- Fixing CFRP strips on tapered-beams, by utilized trowel with light press, and the excess adhesive was removed by trowel. Fig. 3-27.

Fig. 3-27 Fixing of CFRP strip.

3-13 Tests of hardened UHPC

Mechanical properties of UHPC mix were determined, three tests were made: modulus of rupture, splitting tensile, compressive strength, and modulus of elasticity. All the tests were executed according to (ASTM).

3-13-1 Flexural strength

Flexural strength was tested according to (C78/C78M – 15a) [85], three prisms with dimensions (0.1m*0.1m*0.5m) were tested by flexural machine, with 6300N capacity as shown in the Fig. 3-28.

Fig. 3-28 Flexural strength device.

3-13-2 Splitting tensile strength

Splitting tensile is a test to calculate the tensile strength of concrete and it's in according with (C496/C496M – 11) [86], in this test cylindrical concrete of 28-days must utilized with standard dimensions (0.1m diameter, and 0.2m length), and put horizontally in electrical test machine ELE amplitude of 2000 kN as shown in the (Fig. 3-29).

Fig. 3-29 Compressive and splitting tensile strength device.

3-13-3 Compressive strength

Three concrete cubes with dimensions (0.1m*0.1m*0.1m) and three concrete cylinder (0.2 m length and 0.1 m diameter), to determine the compressive strength by utilizing ELE compression machine, its capacity is 2000kN as shown in the (Fig. 3-29).

3-14 Tests of tapered-beams

The tests of all tapered-beams were as simply supported by utilizing two point loads, with utilized steel plate as bearing plate (L= 400 mm, W= 100 mm, H = 80 mm) under each point load. (Fig. 3-30).

Fig.3.30 Bearing plate.

3-14-1 Instruments of test

3-14-1-1 Hydraulic jack with load cell

Hydraulic jack was used to apply loads on the tapered-beam with (60 tons) capacity, and read the value of the applied load by load cell. Tests were carried out at Amarah Technical Institute laboratory. (Fig. 3-31).

Fig. 3-31 Hydraulic jack with load cell.

3-14-1-2 Dial gauge

Dial gauge is measured of deflection. For each loading stride, the load applied reading was recording with the deformation caused by the applied load. (Fig. 3-32) shows dial gage was utilized.

Fig. 3-32 Dial gauge.

3-15 Test setup and instrumentation

The nineteen tapered-beams were tested utilizing a hydraulic machine of 60 ton capacity. All tapered-beams were tested under two point loads utilizing concentrated load on rigid steel I-beam. Applied load was measured, two dial gages were utilized: one at mid span and the other was at (1/3 l_n) from support. (Fig. 3-33) shows the positions of the dial gages.

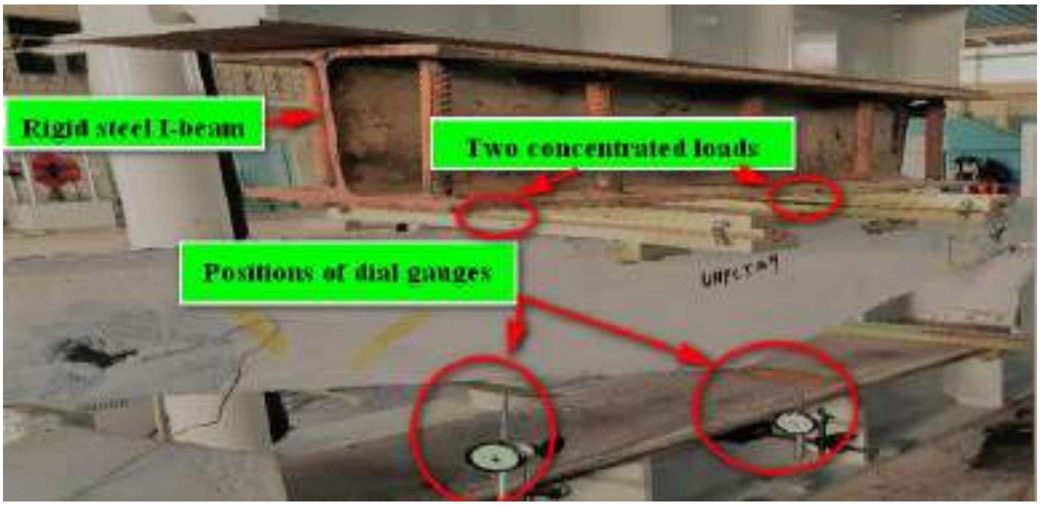


Fig. 3-33 Positions of dial gauges.

CHAPTER FOUR

EXPERIMENTAL RESULTS

4-1 Introduction

The first aim of this study was to develop UHPC from locally available materials, and second to utilize it in the main research (Shear behaviour of UHPC tapered-beams with longitudinal openings). To reach this objective, experimental program was carried out to optimize mix designs with basic available materials in Amara and Baghdad cities. The study was conducted in university of Missan college of Engineering's laboratory and the laboratory of Amarah Technical Institute.

Results had been collected and discussed, load carrying capacity, and deflections were assembled.

Load deflection behaviour for each tested tapered-beam's group was given with its identical sketched picture of tapered-beam during the test.

For tested tapered-beams, sketching had been done on tapered-beam to record first crack, crack pattern and ultimate load in which of applied load occurs, and discussed.

4-2 Experimental results of mix design

4-2-1 Tests of hardened UHPC

4-2-1-1 Flexural strength

The (Fig. 4-1) and Table (4-1) are elucidated results of modulus of rupture of prism. The results of tests were calculated by mathematical formula below that mentioned in (C78/C78M − 15a) [85].

1 kg = 9.8066 N

$$f_r = \frac{3PL}{2bd^2} \qquad \text{....(1) [85]}$$

Where,

(f_r): Flexural strength (modulus of rupture MPa);

(P): Ultimate failure load N;

(L): Clear span c/c 450 mm;

(b): width of prism 100 mm, and;

(d): depth of prism 100 mm.

Fig. 4-1 Results of modulus of rupture for prism.

Table (4-1) Flexural testing results.

Test load kg	2405	2518	2737
Test load N	23584.99	24693.14	26840.8
Stress (f_r) MPa	15.92	16.67	18.12
Average stress MPa	16.9		

4-2-1-2 Splitting tensile strength

Before applying of load two thin pieces of plywood were lay down and top of cylinders to avoid any unacceptable stress concentration as well as compensate for any non-straightening surface as in (Fig. 4-2, and 4-3) Table (4-2) are showed the results of test.

The results of test machine (split tensile strength) based on following formula that mentioned in the (ASTM C496/C496M – 11) [86]:

$$f_t = \frac{2P}{\pi DL} \quad \text{....(2) [86]}$$

Fig. 4-2 Style of splitting testing.

Fig. 4-3 Splitting test result with tensile failure mode.

Table (4-2) Tensile strength testing results.

Test load kN	445.6	445.8	445.9
Tensile strength MPa	14.1	14.1	14.1
Average MPa	14.1		

4-2-1-3 Compressive strength

Compressive strength test of UHPC was according to (ASTM C39/C39M – 15a) [87]. Mixtures compressive strength are measured at 28-days only because of the target was how to get UHPC mixture. The compressive strength data clarified in Table (4-3). And the (Fig. 4-4) shows some of compressive strength reading.

Table (4-3) Test results of compressive strength.

Mix.	Cube compressive strength of 28-Days MPa			Av. MPa	Cylindrical compressive strength of 28-Days MPa			Av. MPa	Difference %
1	107.9	105.9	104	105.9	----	----	----	----	----
2	127.1	123.2	123	124.4	----	----	----	----	----
3	160.8	165.7	167.9	164.8	141.6	148.8	153.9	148.1	0.89
4	123.2	127.1	132.6	127.6	----	----	----	----	----
5	164.7	166.3	170	167	147.3	151.7	153.7	150.9	0.90
6	95.8	101.3	106.8	101.3	----	----	----	----	----
7	125.4	121	130.9	135.7	----	----	----	----	----
8	154.3	165.4	170	163.2	135.3	147.7	155.7	146.2	0.89
9	134.3	144.8	137	138.7	----	----	----	----	----
10	126.4	129.6	133.5	129.8	112.3	115.2	119	115.5	0.889
11	----	----	----	----	145.8	148.2	159.5	151.1	----
12	111.6	113.9	117.7	114.4	----	----	----	----	----
13	123.1	125.4	128.5	125.6	----	----	----	----	----
14	----	----	----	----	160.3	162.8	153.2	110.7	----
15	----	----	----	----	158.5	160.5	161.1	160	----
16	----	----	----	----	72.8	84	79	78.6	----
17	----	----	----	----	77	77.2	78.1	77.4	----
18	104.2	111.9	116.8	110.9	92	99.7	104	98.5	0.88
19	137.6	139.4	142	139.6	123.1	124.7	127	124.9	0.89
20	139	141.2	143.9	141.3	----	----	----	----	----
21	124	126.2	130	126.7	110.6	113	116.5	113.3	0.89
22	----	----	----	----	129.4	125.4	134	129.6	----
23	122.1	125	117	121.3	----	----	----	----	----
24	137.3	140	135	137.4	----	----	----	----	----
25	----	----	----	----	153.6	159.4	156.8	156.6	----
								Av.	0.889

Fig. 4-4 Some of compressive strength readings.

From the results of the hardened tests, it is clear that UHPC mixture can be obtained by using locally available materials and from several types of sand (sand #2, sand #3, and sand #4), not only from the sand that has gradient (0.3-0.6mm) as mentioned in the most of recommendations and researches. Utilizing of sand #5 that has super extra fine gradient leads to decrease compressive strength instead of increases Table (4-3) mixes (16, 17, and 18), this may due to its gradient is less than gradient of cement at sand #5 smallest size and a little more than of cement gradient at sand #5 biggest size, and this not suitable to get UHPC because the mechanism of the particle packing is the silica fume fills the pores between the cement, and the cement fills the pores between the sand not the opposite, so the sand gradient that needs must be at least of its size bigger than cement size to achieve this mechanism, on the other hand the sand#5 increases the water demand due to its gradient. There are several types of plasticizers available in the local markets, but the best of these types is PC260, which has high range reducing of water using in the mixture. The compressive strength is highly depending on the w/c ratio, and a small increasing in the w/c ratio leads to a significant decrease in the compressive strength. The compressive strength depends on size of specimen, the cylindrical compressive strength is less than compressive strength of cubic by 11%.

4-3 Experimental setup

The tapered-beams were tested under two point loads to study the shear behaviour. The applying loads were accomplished utilizing hydraulic jack, and the hydraulic jack was already calibrated to provide required load. Dial gauges were utilized to measure of deflection at two points at mid span and (1/3) of span. Testing of tapered-beams was carried out in the laboratory of Technical Institute of Amarah.

4-3-1 Loading procedure

Loadings were progressively increase from 0 kN to failure with increasing of 10 kN load step as possible. It was choosing to pause nearly (1min.) after each load step to dial gauges measurement and observation (Fig. 4-5). All cracks were disclosed and were marked, and recorded loading value of first crack.

Fig. 4-5 Deflection measurement.

4-3-2 Shear behaviour of tested tapered-beams until failure

The test results' record are given in Table (4-4), this table consists of nineteen simply supported tapered-beam specimens subjected to shear test utilizing two point loads.

(i) Failure modes: the 19 specimens were tested, shear failure was prevailed in all specimens.

(ii) Shear failure's phenomena; Cracks' propagations in inclined direction continue toward tapered-beam's upper edge until one of cracks suddenly is expanded to form critical diagonal shear crack that stops the propagation and formation of other cracks, and eventually causes the brittle failure, and tapered-beams couldn't resist more load.

Load at which tested collapse of tapered-beams described as ultimate load (F_u) or load bearing capacity of tested tapered-beams.

There were observed formation of inclined cracks in tapered-beam were created due to high shear stress especially for tapered-beams haven't shear reinforcement (stirrups), and without CFRP bar/strip.

This failure had inclined crack, and initiated in shear span extended from tensile reinforcement at support toward nearest concentrated load, and intersect the level of tensile reinforcement according to angle of failure. Shear failure is delicate difficult to foretell. Moreover, if tapered-beam without properly designed shear reinforcement (stirrups) may fail by shear, and suddenly occur without warning, except tapered-beam TB 13 in addition of the cracks mentioned above, its failure had inclined crack and initiated in shear span extended from the two concentrated loads toward the bottom edges of NS CFRP bars of tapered-beam in the two sides due to the high confinement in supports' direction that provided by the orientation of NS CFRP bar, also flexural cracks were propagated.

The theoretical shear failure loads had calculated by EXCEL sheets, Appendix (A). So, the longitudinal reinforcement ratio has been considered. Lastly, the type of shear failure was also mentioned.

Table (4-4) Tests results of tested tapered-beams.

Beam ID	Load Characteristics kN		Deflection mm		a/d	Steel fiber %	Inclination Angle (°)	Tensile bars	Steel stirrups for shear span	4 NS CFRP bars, or 4CFRP strips 50mm width	Mode of failure
	First crack	Ultimate	Ultimate deflection	Service deflection							
TB1	202	416	12.1	3.89	2.94	2	15.9°	2Φ25	Without	None	Shear
TB2	147	362	9.3	3.02	2.73	2	9.7°	2Φ25	Without	None	Shear
TB3	164	375	14.94	4.79	2.73	2	12.8°	2Φ25	Without	None	Shear
TB5	187	462	15.5	2.53	2.3	2	15.9°	2Φ25	Without	None	Shear

TB6	221	486	17.59	3.01	2.3	2	15.9°	2Φ25	Without	None	Shear
TB7	162	370	13.11	3.07	2.73	2	15.9°	2Φ16 + 2Φ12	Without	None	Shear
TB8	205	446	18.49	3.9	2.73	2	15.9°	4Φ16	Without	None	Shear
TB9	186	460	15.45	3.5	2.3	2	15.9°	2Φ25	Without	None	Shear
TB10	183	432	17.35	2.05	2.73	2	15.9°	2Φ25	Without	None	Shear
TB11	189	481	19.35	4.32	2.73	2	15.9°	2Φ25	without	NS 0°	Shear
TB12	288	590	34.52	4.61	2.73	2	15.9°	2Φ25	without	NS 45°	Shear
TB13	263	585	32.2	4.55	2.73	2	15.9°	2Φ25	without	NS 30°	Shear
TB14	176	441	11.2	3.6	2.73	2	15.9°	2Φ25	without	strip 0°	Shear
TB15	205	487	17.81	2.66	2.73	2	15.9°	2Φ25	without	strip 45°	Shear
TB16	188	465	11.24	4.39	2.73	2	15.9°	2Φ25	without	strip 30°	Shear
TB17	263	565	27.15	4.92	2.73	2	15.9°	2Φ25	5Φ8mm	None	Shear
TB18	244	518	22.5	5.65	2.73	2	15.9°	2Φ25	4Φ8mm	None	Shear
TB19	111	111	7.85	-----	2.73	0	15.9°	4Φ16	without	None	Shear
TB20	123	281	13.03	3.21	2.73	1	15.9°	4Φ16	without	None	Shear

4-3-3 Results of measurements

Results of deflection were measured by utilized two dial gauges, as presented before in Chapter 3, load deflection relations of tapered-beams were recorded constantly during testing. Results were utilized to analyze main behavior of tapered-beams up to failure as well as to detect similarities and variations of all of tested tapered-beams. The results of load deflection relation for all groups are showed that when load increases the deflection increases, by another word the tapered-beam that has the greater shear failure load in the group has the greater deflection and this applies and goes for all groups, and this due to the directly relationship between the load and deflection.

4-3-3-1 First group (inclination angle)

4-3-3-1-1 Deflection

The Fig. 4-6 and Table (4-5) are elucidated mensuration of load deflection relation result for first group tested tapered-beams. It's noticed that tapered-beam TB 10 with 15.897° inclination angle, and has two openings has greater deflection by 86.5 % and 16.1% than TB 2 and TB 3 respectively with inclination angle 12.835°, and 9.697° and have two openings, and this due to the increasing of inclination angle means increasing the total depth for that the shear capacity and deflection of tapered-beam increase.

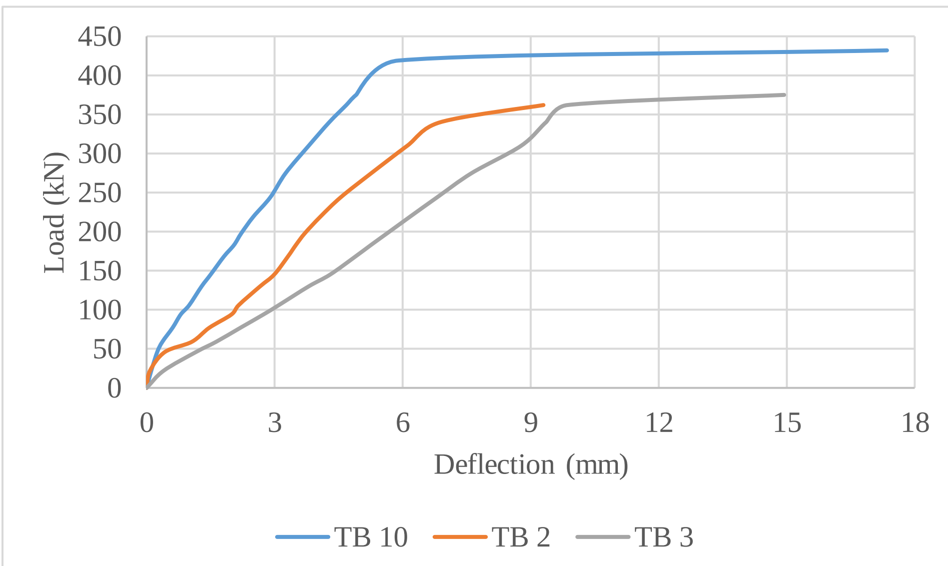


Fig. 4-6 Load deflection relation for first group.

Table (4-5) The difference in load and deflection for first group.

Beam ID	Inclination angle	First crack Load kN	Difference %	Ultimate Load kN	Difference %	Ultimate Deflection mm	Difference %
TB 10	15.9˚	183	---	432	---	17.35	---
TB 3	12.8˚	164	-11.6	375	-15.2	14.94	-16.1
TB 2	9.7˚	147	-24.5	362	-19.3	9.3	-86.5

4-3-3-1-2 Effect of inclination angle

The Influence of the inclination angle on the shear strength capacity of tapered-beams is shown in (Fig. 4-7). The inclination angle induces comparatively positive effects on shear strength capacity of tapered-beams. The interpretation of this situation can be explained as: the vertical component of the tensile-stress on the longitudinal reinforcement give rise to positive effect on shear capacity of the tapered-beams due to its directions, and this clearly from results of test that inclinations have a powerful impact on shear behavior as well as shear capacity of concrete tapered-beam hasn't stirrups, which means inclined angles induces comparatively positive impacts to increase the shear strength of tapered-beams, in other words: the shear capacity of tapered-beam increases as the inclined angle increases, when angle varied from 9.7° to 15.9° the failure load, first crack load, and deflection, were increased by (19.3%, 24.5%, and 86.5%) respectively. (Figs. 4-6 & 4-7) and Tables (4-4 and 4-5).

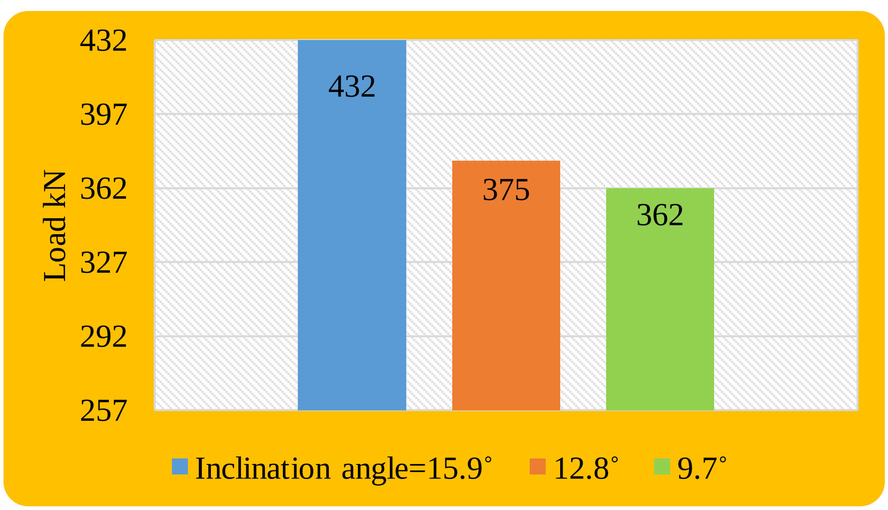


Fig. 4-7 Failure load versus inclined angle.

4-3-3-2 Second group (Openings)

4-3-3-2-1 Deflection

The Fig. 4-8 and Table (4-6) are elucidated mensuration of load deflection relation result for second group tested tapered-beams. It's noticed that tapered-beam TB 6 with 15.897° inclination angle and one opening in the tapered zone H2 has greater deflection by 13.5% and 13.8% than TB 5 and TB 9 respectively those have same inclination angle, and with one opening in the prismatic zone H1, and two openings in the prismatic and tapered zone H1 + H2 respectively. This indicates that concrete core in tapered-beam TB 6 with solid prismatic zone participates in increasing deflection due to increasing shear capacity. The deflection values of TB 5 and TB 9 were very close, due to the convergence of their shear failure loads.

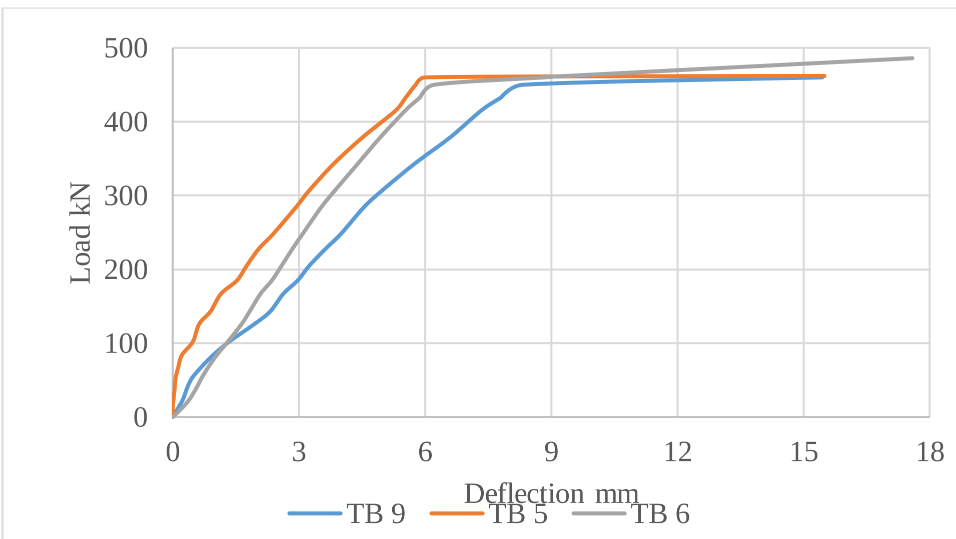


Fig. 4-8 Load deflection relation for second group.

Table (4-6) The difference in load and deflection for second group.

Beam ID	Openings No. and position	First crack Load kN	Difference %	Ultimate Load kN	Difference %	Ultimate Deflection mm	Difference %
TB6	1 H2	221	---	486	---	17.59	---
TB5	1 H1	187	- 18.2	462	- 5.2	15.5	- 13.5
TB9	2(H1+H2)	186	-18.8	460	- 5.6	15.45	- 13.8

4-3-3-2-2 Effect of openings

From (Fig. 4-9) the openings in the prismatic zone of tapered-beams contributed in decreasing of load carrying capacity, first crack load, deflections, and service deflection by (5.6%, 18.8%, 13.8%, and 19%) respectively (Fig. 4-8) and (Fig. 4-9) shows failure load versus location of openings. This indicates that concrete core in tapered-beam with solid prismatic zone participates in increasing load capacity and deflection. The tapered-beam with one opening in prismatic region has the same shear capacity and deflection of the tapered-beam with two openings; because the tapered opening is out of failure region, so the possibility of using two logical holes in each prismatic and non-prismatic region is more economic and lightly.

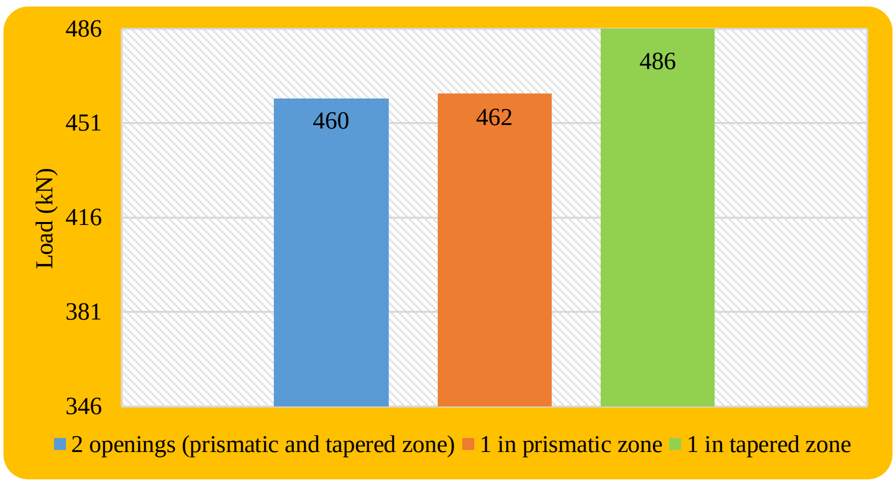


Fig. 4-9 Failure load versus position of openings.

4-3-3-3 Third group (longitudinal reinforcement ratio)

4-3-3-3-1 Deflection

The Fig. 4-10 and Table (4-7) are elucidated mensuration of load deflection relation result for third group tested tapered-beams. It's noticed that tapered-beam TB 8 with 15.897° inclination angle, has two openings, and 4Φ 16mm in two rows has greater deflection by 6.6% and 41% than TB 10 and TB 7 respectively with same inclination angle, and with 2Φ25mm and (2Φ 16mm, and 2Φ12mm) in 2-rows respectively. This may due to the dowel action effect is greater when big bars of tensile reinforcement distribution in two rows.

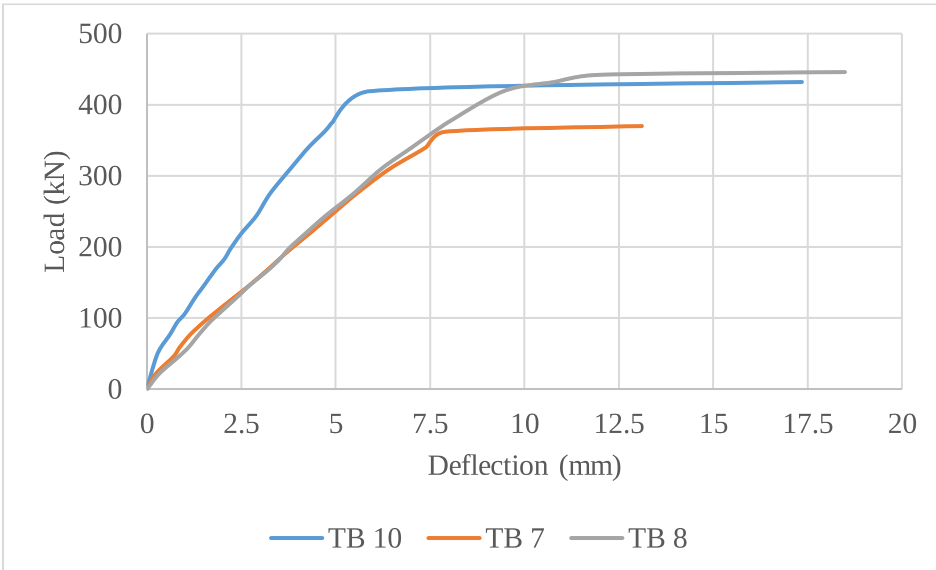


Fig. 4-10 Load deflection relation for third group.

Table (4-7) The difference in load and deflection for third group.

Beam ID	Longitudinal reinforcement	First crack Load kN	Difference %	Ultimate Load kN	Difference %	Ultimate Deflection mm	Difference %
TB8	4Φ16	205	---	446	---	18.49	---
TB 10	2Φ25	183	- 12	432	- 3.2	17.35	- 6.6
TB7	2Φ16 + 2Φ12	162	- 26.5	370	- 20.5	13.11	- 41

4-3-3-3-2 Effect of longitudinal reinforcement ratio

(Fig. 4-11) presents the effect of steel reinforcement ratio ($\rho\ell$) of tested (UHPC) tapered-beams without stirrups on shear strength. It is evident that increasing of ($\rho\ell$) led to increase in shear strength due to increase the dowel action component, in same time if main reinforcement distributed in two rows gives more effect on shear strength even if its steel area was less than one row steel's area. When steel area increased from (628.3 mm² 1.22%) to (804.2 mm² 1.57%) (tensile bars were distributed by two rows for each), the ultimate load, first crack load, service deflection, and deflection increased by (20.5%, 26.5%, 27%, and 41%) respectively. When steel's area increased from (628.3mm² in two rows 1.22%) to (981.7mm² in one row 1.79%) shear capacity increased by 16.7%, also the first cracking load, and deflection increased by (12.9%, and 32.3%) respectively this decreasing due to utilized small bars (12 mm) in the second row. But when steel area decreased from (981.7mm² in one row 1.79%) to (804.2mm² in two rows 1.57%) shear capacity increased by (3.2%) despite of steel area was lesser by (18%), but distributed by two rows, also the first cracking load, service deflection, and deflection increased by (12%, 90.2%, and 6.6%) respectively, due to the dowel action effects when tensile bars distributed by two rows instead of one row even when utilized lesser area in case of two rows, this means the dowel action effect increase if tensile bar distributed by more one row with utilized big bars (Figs. 4-10 & 4-11) and (Table 4-4).

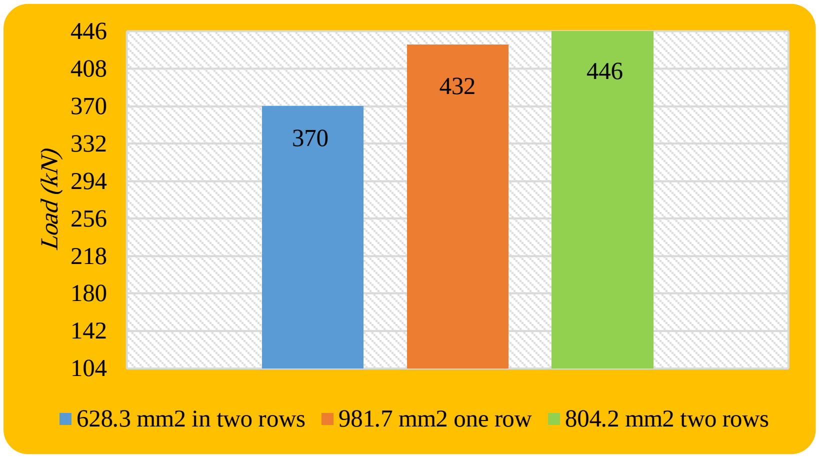


Fig. 4-11 Failure load versus area of tensile bars.

4-3-3-4 Fourth group (shear span to effective depth a/d ratio)

4-3-3-4-1 Deflection

The Fig. 4-12 and Table (4-8) are elucidated mensuration of load deflection relation result for fourth group tested tapered-beams. It's noticed that the deflection of TB10 is greater than of TB1 and TB 9 by 12.3% and 43.4% respectively. Generally the greater deflection is associated with the higher load, but here TB 9 has greater load, but has lower deflection of TB 10 because it has the lesser a/d which means the applied load is closer to the support than TB10, and the deflection will smaller whenever the load approached from the support. For TB 1 that has the lowest deflection and in the same time has the greater a/d (which means closer to the mid span and farther from support), this due to TB 1 has the lowest failure load.

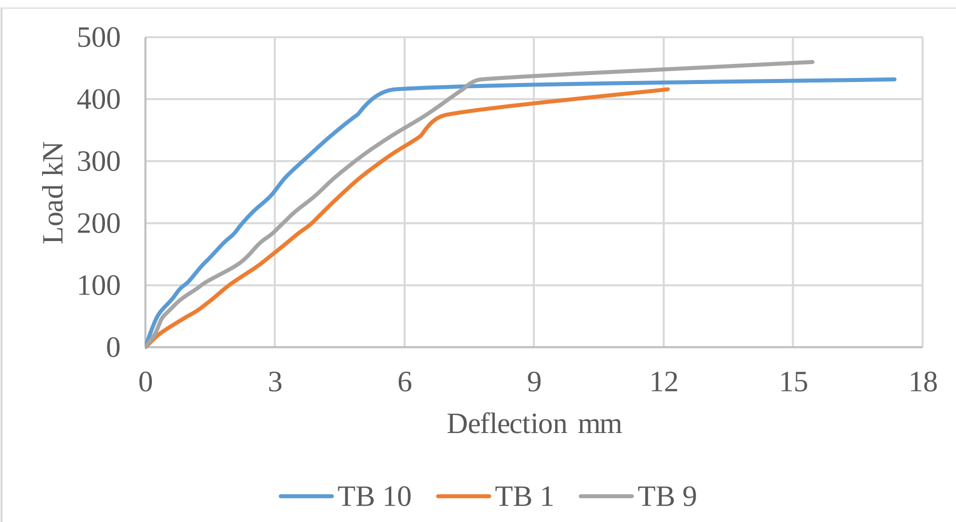


Fig. 4-12 Load deflection relation for fourth group.

Table (4-8) The difference in load and deflection for fourth group.

Beam ID	a/d	First crack Load kN	Difference %	Ultimate Load kN	Difference %	Ultimate Deflection mm	Difference %
TB 10	2.73	183	---	432	---	17.35	---
TB1	2.94	202	10.4	416	- 3.8	12.1	- 43.4
TB9	2.3	186	1.6	460	6.5	15.45	- 12.3

4-3-3-4-2 Effect of shear span to effective depth (a/d) ratio

Generally, shear strength of tapered-beams decreases when a/d ratio increases, (Fig. 4-13) shows the UHPC tapered-beams' ultimate shear failure loads which decreased when a/d ratio increasing.

In those tapered-beams, as formerly stated, some diagonal cracks were directly generated in shear span.

Ultimate shear failure loads of UHPC tapered-beams decrease by increasing of a/d ratio, when a/d decreasing from 2.94 to 2.3 led to increase the failure loads and deflection about (10.6%, and 27.7%) respectively due to the relation between a/d and shear capacity (*a/d = M / V d)*, but decreasing in first crack load, and service deflection by (7.9%, and 10%) respectively.

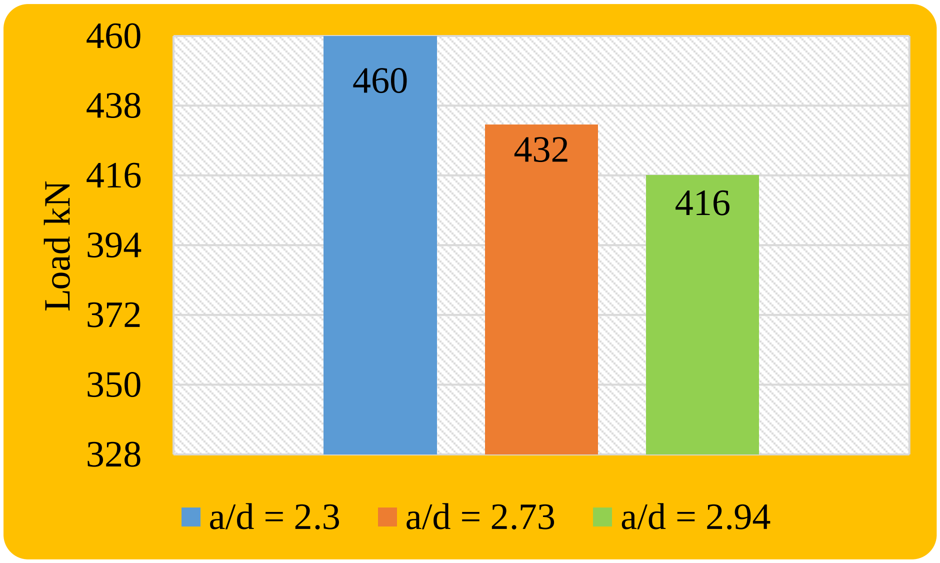


Fig. 4-13 Failure load versus (a/d).

4-3-3-5 Fifth group (NS CFRP bars)

4-3-3-5-1 Deflection

The Fig. 4-14 and Table (4-9) are elucidated mensuration of load deflection relation result for fifth group tested tapered-beams. It's noticed that tapered-beam TB 12 with 15.897° inclination angle, two openings, and with

4 NS CFRP bars Φ 6 mm in 45° orientations, has greater deflection by 78.4%, and 7.2% than TB 11 and TB 13 respectively those have the same TB 12 (inclination angle, openings, and NS CFRP bar), but with (0°, and 30° NS CFRP orientation) respectively, and TB 12 has greater deflection than TB 10 by 99%. This is due to the inclination angle of NS CFRP that caused increases the shear capacity of tapered-beam due to the high confinement, therefore increases the deflection. The utilized NS CFRP bar with angles 30°, and 45° in shear zone leads to increase the deflection of tapered-beam.

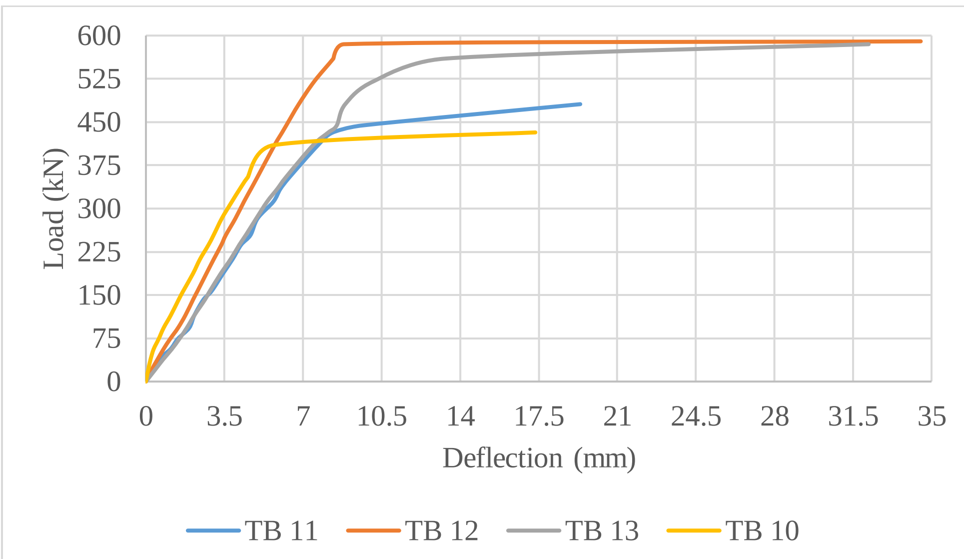


Fig. 4-14 Load deflection relation for fifth group.

Table (4-9) The difference in load and deflection for fifth group.

Beam ID	4 NS CFRP bars Φ 6mm	First crack Load kN	Difference %	Ultimate Load kN	Difference %	Ultimate Deflection mm	Difference %
TB 10	without	183	---	432	---	17.35	---
TB 11	0˚	189	3.3	481	11.3	19.35	11.5
TB 12	45˚	288	57.4	590	36.6	34.52	99
TB 13	30˚	263	43.7	585	35.4	32.2	85.6

4-3-3-5-2 Effect of NS CFRP bars

Shear capacity of tapered-beam increases by (11.3%, 35.4 %, 36.6 %) when utilized NS CFRP bar with orientations (0˚, 30°, 45°) respectively, and the first cracking load increased by (3.3%, 43.7%, and 57.4%) respectively, service deflection increases by (110.7%, 121.9%, and 124.9%) respectively, and deflection increased by 11.5%, 85.6%, and 99% respectively (Figs. 4-14, 4-15, and 4-16) and (Table 4-4). It's verified that inclined NS CFRP bar are more efficient than vertical ones this may due to the high confined that resulted from the inclined angle of NS versus angle of crack. Utilizing of NS CFRP bar led to increase the shear capacity of tapered-beam.

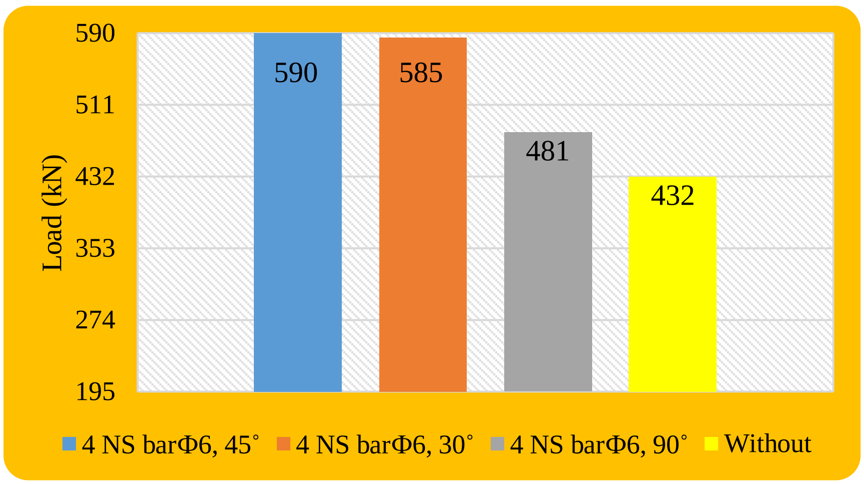


Fig.4.15 Contribution of NS CFRP bars in shear capacity.

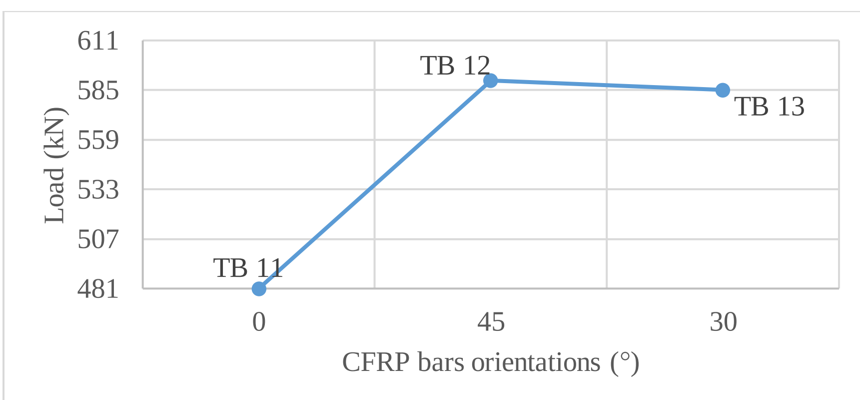


Fig. 4-16 Failure load versus NS SFRP bars orientation.

4-3-3-6 Sixth group (CFRP strips)

4-3-3-6-1 Deflection

The Fig. 4-17 and Table (4-10) are elucidated mensuration of load deflection relation result for sixth group tested tapered-beams. It's noticed that tapered-beam TB 15 with 15.897° inclination angle, two openings, and with 4 CFRP strips 50 mm width in 45° orientations, has the greater deflection by 59% and 58.4% than TB 14 and TB 16 respectively those with same TB 15 inclination angle, openings, and CFRP strips, but with (0°, and 30° CFRP orientation) respectively. This due to the inclination angle of CFRP strips. The utilized CFRP strip with an angle 45° in shear zone leads to increase the deflection of tapered-beam.

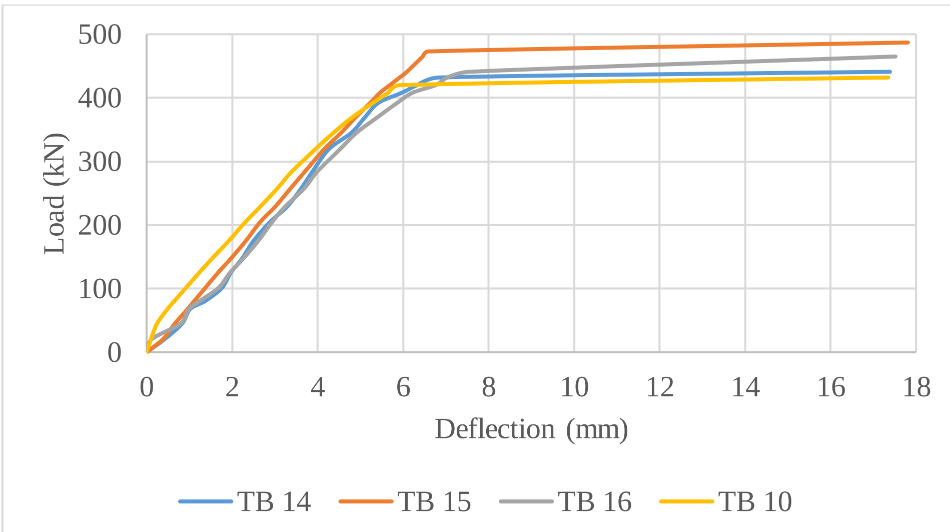


Fig. 4-17 Load deflection relation for sixth group.

Table (4-10) The difference in load and deflection for sixth group.

Beam ID	4 CFRP strip 50mm width	First crack Load kN	Difference %	Ultimate Load kN	Difference %	Ultimate Deflection mm	Difference %
TB 10	without	183	---	432	---	17.35	---
TB 14	0˚	176	- 3.8	441	2	11.2	- 54.9
TB 15	45˚	205	12	487	12.7	17.81	2.6
TB 16	30˚	188	2.7	465	7.6	11.24	- 54.3

4-3-3-6-2 Effect of CFRP strips

The shear capacity of tapered-beam increases by 2%, 7.6%, and 12.7% when utilized CFRP strip with orientation 0˚, 30˚, and 45° respectively comparing with TB 10, for CFRP strips with 45˚ orientations, first cracking load, service deflection, and deflection increased by 12%, 29.7%, and 2.6% respectively. (Figs. 4-17, 4-18, and 4-19) and Tables (4-4 and 4-10). It's verified that inclined CFRP strips are more efficient than vertical ones.

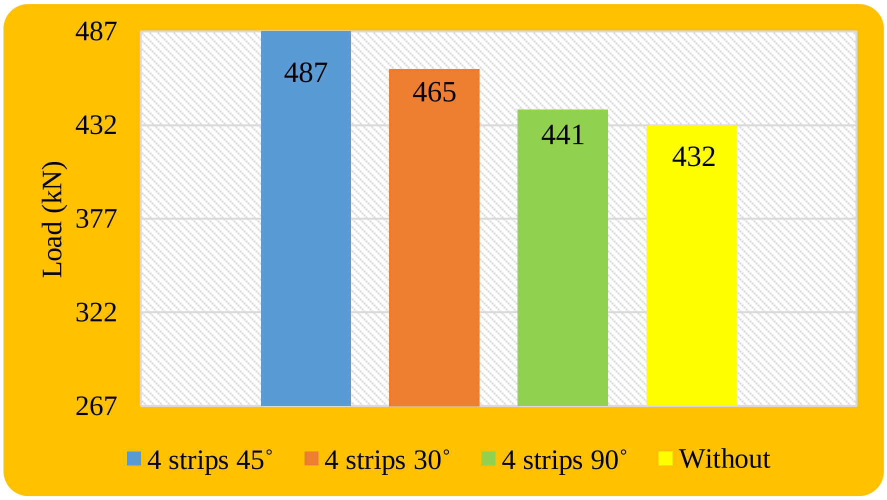


Fig.4.18 Contribution of CFRP strips in shear capacity.

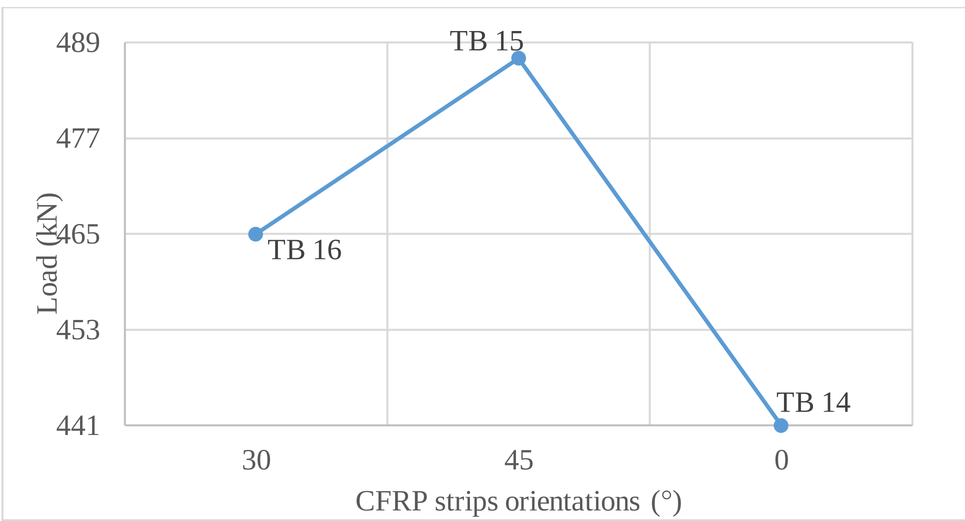


Fig. 4-19 Failure load versus CFRP strips orientation.

4-3-3-7 Seventh group (shear reinforcement (stirrups))

4-3-3-7-1 Deflection

The Fig. 4-20 and Table (4-11) are elucidated mensuration of load deflection relation result for seventh group tested tapered-beams. It's noticed that tapered-beam TB 17 with 15.897° inclination angle, two openings, and with 5 Φ 8 mm stirrups, has greater deflection by 56.5% and 20.7% than TB 10 without stirrups, and TB 18 with 4 Φ 8 mm respectively. This due to higher difference in ultimate strength between tapered-beams that provided by stirrups due to confinement. The utilizing of the steel stirrups lead to increase the deflection of the tapered-beam.

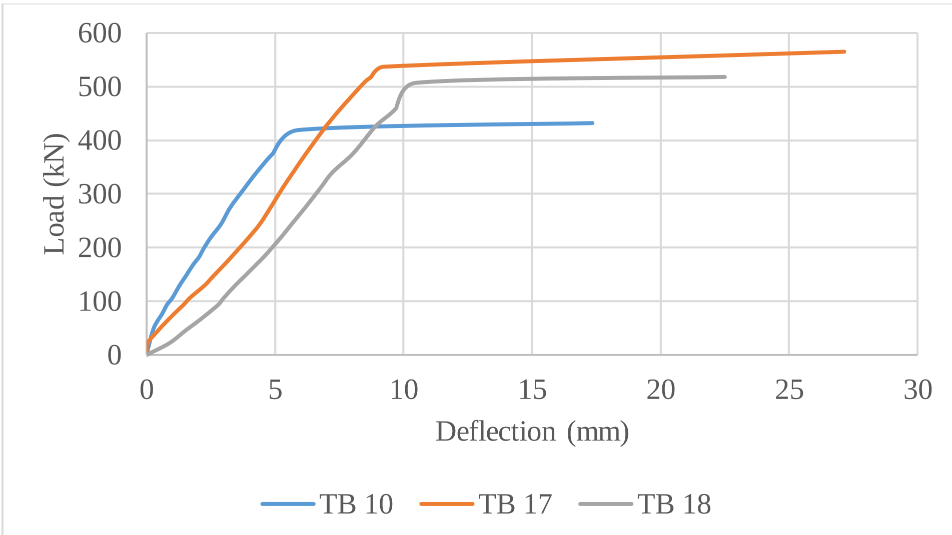


Fig. 4-20 Load deflection relation for seventh group.

Table (4-11) The difference in load and deflection for seventh group.

Beam ID	Stirrups	First crack Load kN	Difference %	Ultimate Load kN	Difference %	Ultimate Deflection mm	Difference %
TB 10	without	183	---	432	---	17.35	---
TB 17	5 Φ 8	263	43.7	565	30.8	27.15	56.5
TB 18	4 Φ 8	244	33.3	518	19.9	22.5	29.7

4-3-3-7-2 Effect of shear reinforcement (stirrups)

Generally, the presence of stirrups enable the transfer of tensile actions across inclined shear cracks, and confining the concrete compression zone, thus increasing the shear capacity. Shear capacity, first crack load, service deflection, and deflection of tapered-beams increases by (30.8%, 43.7%, 140%, and 56.5%) respectively, when transverse reinforcement (stirrups) increased from (0 for TB 10 to 5 stirrups for TB 17). The shear capacity of tapered-beam increases by (19.9%) when stirrups number increased from (0 to 4 stirrups), and the first cracking load, service deflection, and deflection increased by (33.3%, 175.6%, and 29.7%) respectively, (Figs. 4-20, 4-21 & 4-22) and (Table 4-4). The shear strength increases as number of stirrups increase this due to increase the compressive zone confinement that provided by stirrups and transfer more tensile stress by increasing number of stirrups.

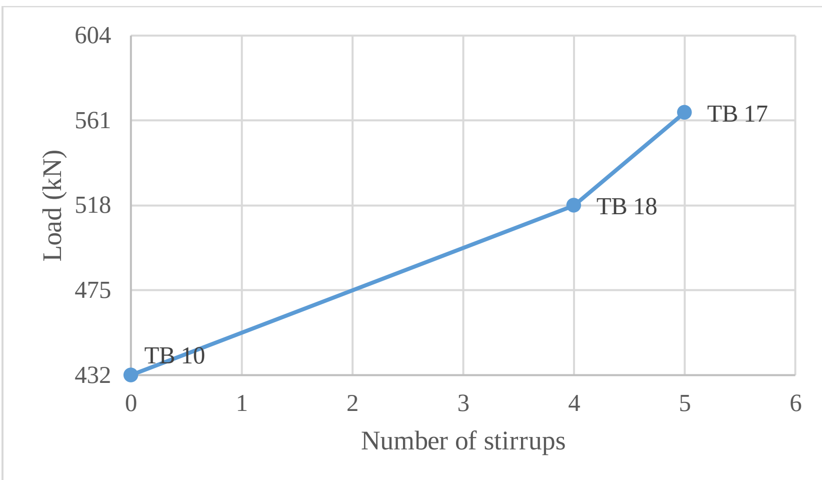


Fig. 4-21 Failure load versus number of stirrups.

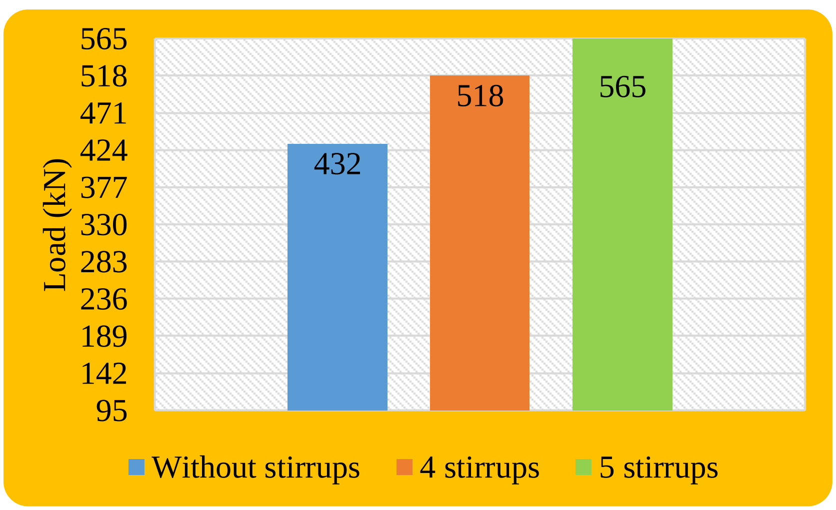


Fig. 4-22 Contribution of stirrups in shear capacity.

4-3-3-8 Eighth group (steel fiber ratio)

4-3-3-8-1 Deflection

The Fig. 4-23 and Table (4-12) are elucidated mensuration of load deflection relation result for eighth group tested tapered-beams. It's noticed that tapered-beam TB 8 with (15.897°) inclination angle, two openings, without stirrups, 4 Φ 16 in two rows, and steel fiber by 2%, has greater deflection by 135.5% and 41.9% than TB 19 and TB 20 respectively those have the same properties of TB 8 except steel fiber ratio 0% and 1% respectively. This due to the steel fiber works as bridges those connect the concrete block and this gives more shear strength to the tapered-beam and increases the deflection. The utilizing of steel fiber leads to increase the deflection of tapered-beam.

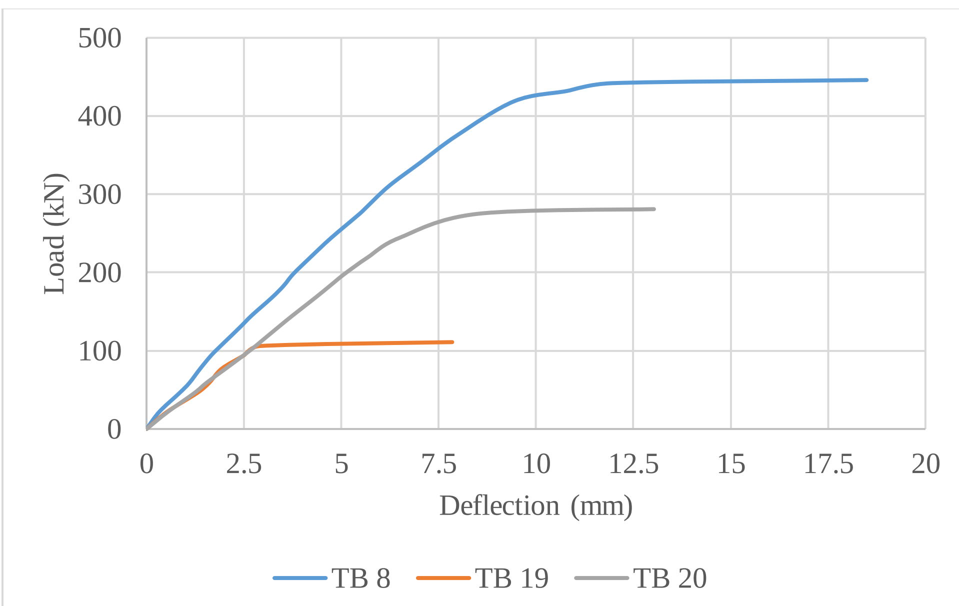


Fig. 4-23 Load deflection relation for eighth group.

Table (4-12) The difference in load and deflection for eighth group.

Beam ID	Steel fiber ratio %	First crack Load kN	Difference %	Ultimate Load kN	Difference %	Ultimate Deflection mm	Difference %
TB 8	2	205	---	446	---	18.49	---
TB 20	1	123	- 66.7	281	- 58.7	13.03	- 41.9
TB 19	0	111	- 84.7	111	- 301.8	7.85	- 135.5

4-3-3-8-2 Effect of steel fiber ratio

For steel fiber's effect on shearing strength, it's clearly from (Figs. 4-24 and 4-25) that 2% by volume of steel fibers are increasing tapered-beam's shear capacity of TB 8 by (300 % which means three times) compared with tapered-beam TB 20 with (0% steel fibers). It’s increasing in the value of load at which first crack appeared 84.7%, and increased the deflection by 135.5%, when steel fibers didn’t existence suddenly failure happened without warning (failure happened in the same time crack’s initiating) (Figs. 4-24, and 4-25) and (Table 4-4). The reason is that the steel fiber works as bridges that connect the concrete block during the applied of loads, after the initiating of cracks in the concrete, the stresses are transfer through steel fibers those crosses the cracks and working as a bridge.

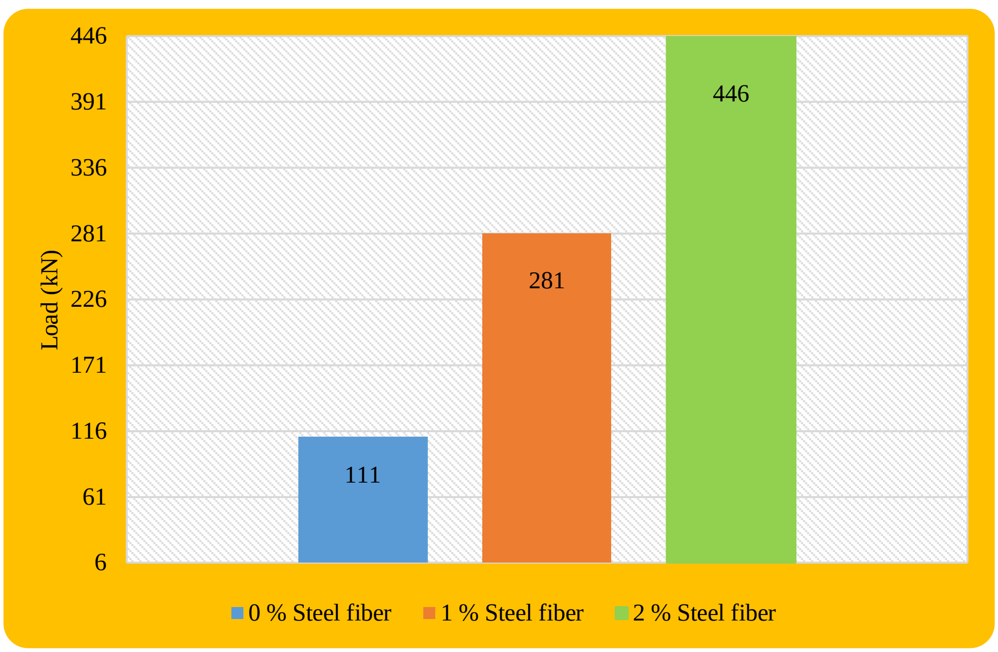


Fig. 4-24 Contribution of steel fiber in shear capacity.

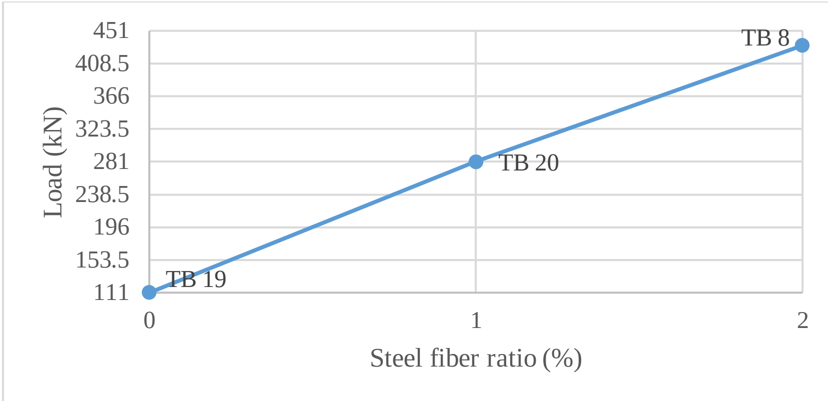


Fig. 4-25 Failure load versus steel fiber ratio.

4-3-3-9 Ninth group (comparison between CFRP bars and strips)

4-3-3-9-1 Deflection

The Fig. 4-26 and Table (4-13) are elucidated mensuration of load deflection relation result for ninth group tested tapered-beams. It's noticed that tapered-beams those strengthen with NS CFRP bars TB 11, TB 12, and TB 13, with orientation 0°, 45°, 30° respectively have greater deflection by 72.8%, 93.8%, and 186.5% than TB 14, TB 15, and TB 16 respectively those have same properties, but with 0°, 45°, and 30° CFRP strips orientation respectively. This due to the ultimate strength that provided by NS CFRP bars that resulted by high confinement in the compressive zone better than CFRP strips, and this caused increased in both shear capacity and deflection.

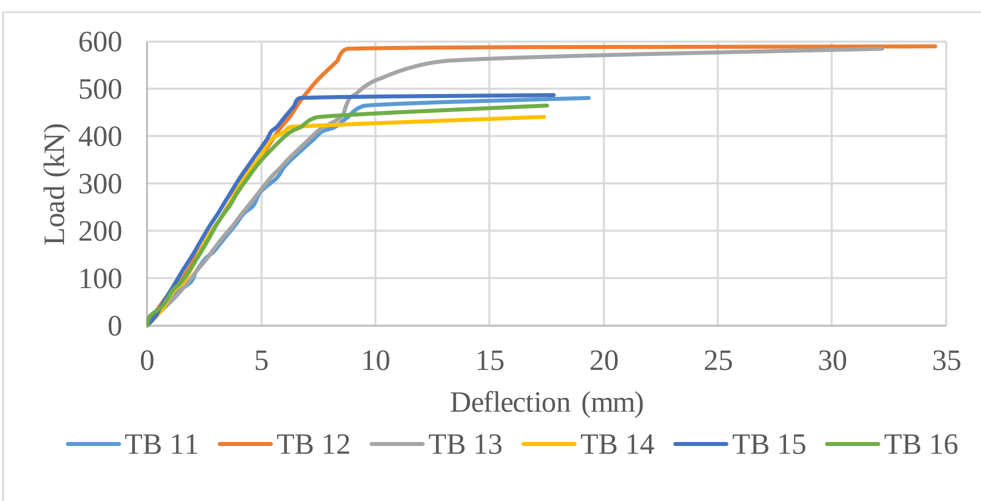


Fig. 4-26 Load deflection relation for ninth group.

Table (4-13) The difference in load and deflection for ninth group.

Beam ID	NS CFRP bar or CFRP strip	First crack Load kN	Difference %	Ultimate Load kN	Difference %	Ultimate Deflection mm	Difference %
TB 10	Without	183	---	432	---	17.35	---
TB 11	NS 0˚	189	3.3	481	11.3	19.35	11.5
TB 12	NS 45˚	288	57.4	590	36.6	34.52	99
TB 13	NS 30˚	263	43.7	585	35.4	32.2	85.6
TB 14	Strip 0˚	176	- 3.9	441	2.1	11.2	- 54.9
TB 15	Strip 45˚	205	12	487	12.7	17.81	2.6
TB 16	Strip 30˚	188	2.7	465	7.6	11.24	- 54.3

4-3-3-9-2 Comparison between NS CFRP bars and CFRP strips

Shear capacity of tapered-beams with NS CFRP bar is more aptitude than CFRP strip in all orientations in increasing of ultimate load capacity, first cracking load, service deflection, and final deflection by (21.1%, 40.5%, 73.3%, and 93.8%) respectively for 45˚ orientations (Figs. 4-26, 4-15, 4-18, 4-16, 4-19, and 4-27) and Tables (4-4 and 4-13). The slightly effect of CFRP strips on shear strength, in comparison with NS CFRP bars, this may due to the type of technique that utilized (U-shape), and if it was with fully wrapping the effect would have been clearer.

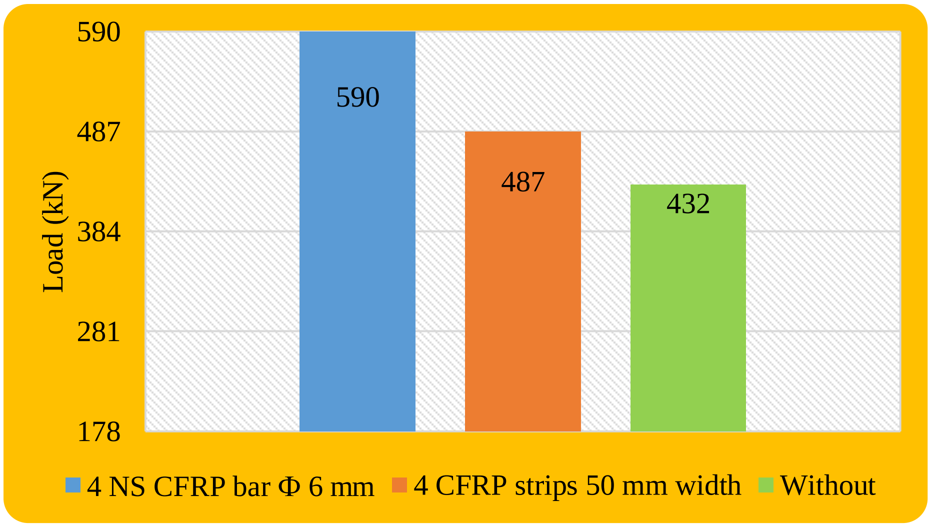


Fig. 4-27 NS CFRP bars versus CFRP strips in shear capacity.

4-3-3-10 Tenth group (comparison between NS CFRP & stirrups)

4-3-3-10-1 Deflection

The Fig. 4-28 and Table (4-14) are elucidated mensuration of load deflection relation result for tenth group tested tapered-beams. It's noticed that tapered-beam TB 12 and TB 13 with 15.897° inclination angle, two openings, steel fiber ratio 2%, and with 4 NS CFRP bars Φ 6 mm in 45° and 30° orientations respectively, have greater deflections by 53.4% and 43.1% respectively than TB 18 that with same properties of TB 12 and TB 13, but hadn't NS CFRP and had 4 Φ 8 mm stirrups. Also the TB 12 and TB 13 have deflection greater than TB 17 by 27% and 18.6% respectively. Even though TB 17 have the same properties of TB 12, and TB 13, but without NS CFRP bars, and with 5 stirrups Φ 8 mm, but its deflection lesser. This is due to the difference in ultimate strength that increased by utilized inclined NS CFRP bar that caused higher deflection.

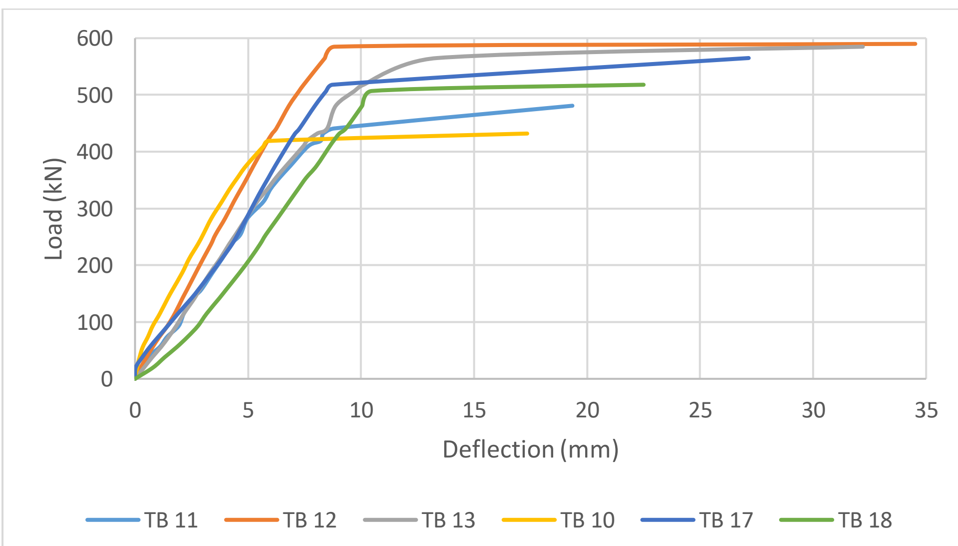


Fig. 4-28 Load deflection relation for tenth group.

Table (4-14) The difference in load and deflection for tenth group.

Beam ID	NS CFRP bar or stirrups	First crack Load kN	Difference %	Ultimate Load kN	Difference %	Ultimate Deflection mm	Difference %
TB 10	Without	183	---	432	---	17.35	---
TB 11	4 NS 0˚	189	3.3	481	11.3	19.35	11.5
TB 12	4 NS 45˚	288	57.4	590	36.6	34.52	99
TB 13	4 NS 30˚	263	43.7	585	35.4	32.2	85.6
TB 18	4 Φ 8	244	33.3	518	19.9	22.5	29.7
TB 17	5 Φ 8	263	43.7	565	30.8	27.15	56.5

4-3-3-10-2 Comparison between NS CFRP bars & stirrups

The utilizing of deformed NS CFRP bars with 45° orientations angle in tapered-beam showed more effective of stirrups in increasing ultimate load, first cracking load, and deflection by (13.9%, 18%, and 53.4%) respectively, comparing with the same number of bars instead of the stirrups diameter was 8 mm and NS CFRP deformed bar was 6 mm, and this is also a point in favor of NS bar, this could be due to the high tensile strength of the CFRP bar, which is far greater than the tensile strength of the steel bar and this with inclined angle provide more confinement and more transfer of tensile action (Figs. 4-28 and 4-29) and (Table 4-4).

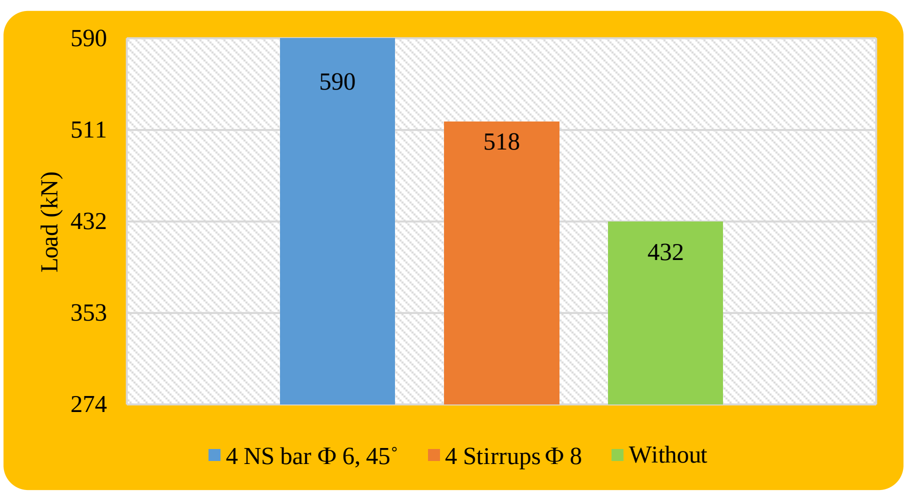


Fig. 4-29 NS CFRP bar versus steel stirrups in shear capacity.

4-3-3-11 Eleventh group (CFRP strips & stirrups)

4-3-3-11-1 Deflection

The Fig. 4-30 is elucidated mensuration of load deflection relation result for eleventh group tested tapered-beams. It's noticed that tapered-beam TB 17 with 5 stirrups Φ 8 mm has higher deflection as elucidated in the Table (4-15). This is due to TB 17 has the maximum number of stirrups and it is provided higher confinement to support compressive zone and transfers more tensile stress that generated because compressive forces (applying and reaction) in the concrete zone, and this led to increase deflection than the CFRP strips technique.

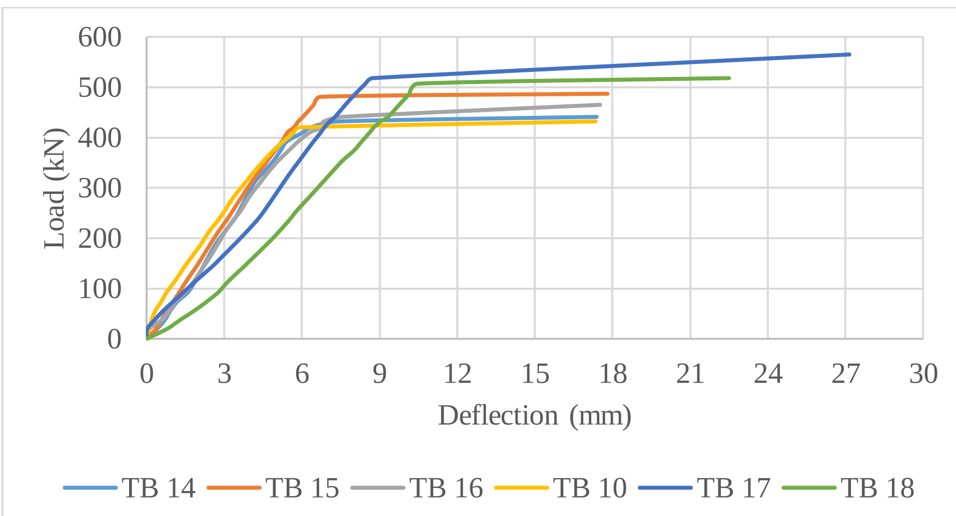


Fig. 4-30 Load deflection relation for eleventh group.

Table (4-15) The difference in load and deflection for eleventh group.

Beam ID	CFRP strips or stirrups	First crack Load kN	Difference %	Ultimate Load kN	Difference %	Ultimate Deflection mm	Difference %
TB 10	Without	183	---	432	---	17.35	---
TB 14	4 Strip 0˚	176	- 3.9	441	2.1	11.2	- 54.9
TB 15	4 Strip 45˚	205	12	487	12.7	17.81	2.6
TB 16	4 Strip 30˚	188	2.7	465	7.6	11.24	- 54.3
TB 18	4 Φ 8 mm	244	33.3	518	19.9	22.5	29.7
TB 17	5 Φ 8 mm	263	43.7	565	30.8	27.15	56.5

4-3-3-11-2 Comparison between CFRP strips & stirrups

steel stirrups showed more effective of CFRP strip with 45˚ orientations for ultimate load, service load, service deflection, and deflection by (6.3%, 19%, 112.4%, and 26.3%) respectively (Figs. 4-30, and 4-31) and Tables (4-4 and 4-15), this may due to the implementation of the CFRP strips with U-wrapping, and if it was by fully wrapping the shear capacity might have been higher.

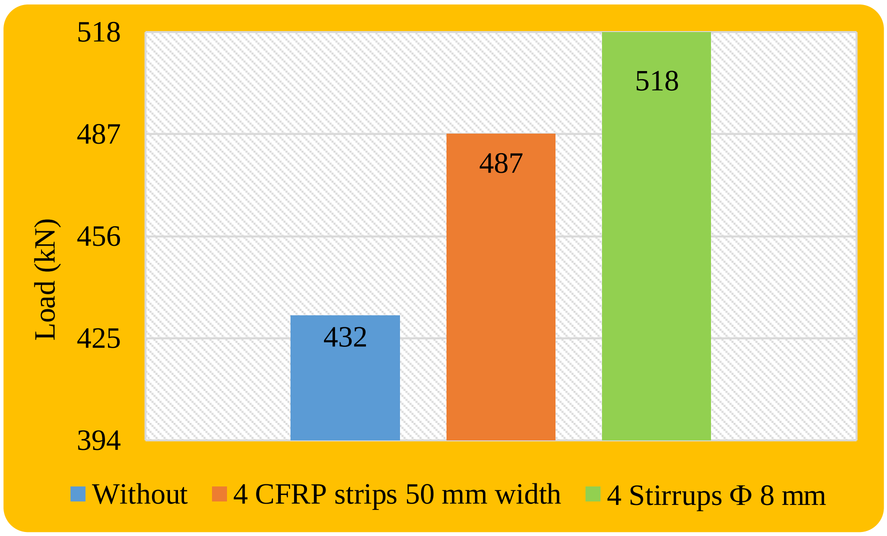


Fig. 4-31 CFRP strips versus steel stirrups in shear capacity.

4-3-3-12 Twelfth group (steel fiber and stirrups)

4-3-3-12-1 Deflection

The Fig. 4-32 elucidate mensuration of load deflection relation result for twelfth group tested tapered-beams. From Table (4-11) and from group seven the tapered-beam TB 17 with 5 Φ 8 mm stirrups has the greater deflection due to the stirrups effect by 56.5%. And from Table (4-12) and from group eight the tapered-beam TB 8 with 2% steel fiber has the greater deflection due to the effect of steel fiber by 135.5% Table (4-16). This means the deflection that provided by steel fiber is more than of stirrup. This may due to the bridges those provided by steel fiber to transfer the stresses and connected the concrete block is more than the confinement and tensile stress that transfer by stirrups.

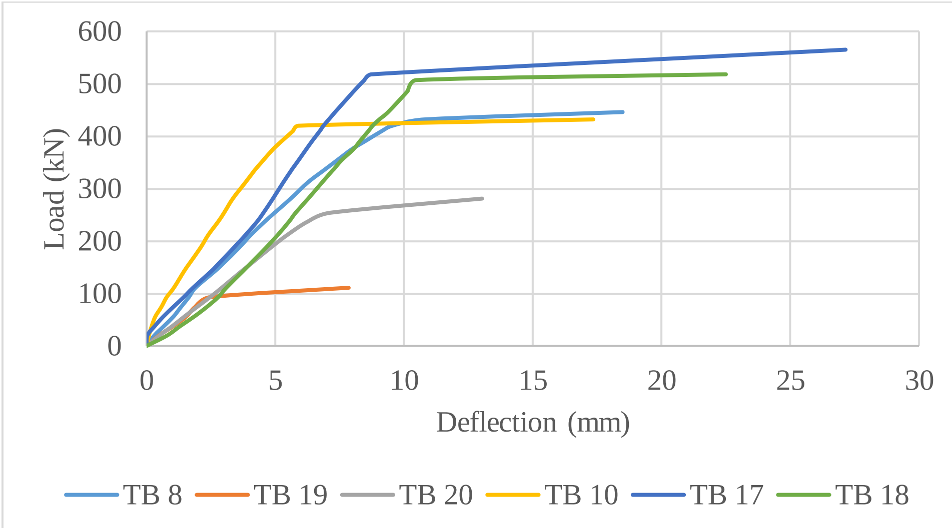


Fig. 4-32 Load deflection relation for twelfth group.

Table (4-16) The difference in load and deflection for twelfth group.

Beam ID	Steel fiber %	Stirrups Φ 8 mm	First crack Load kN	Difference %	Ultimate Load kN	Difference %	Ultimate Deflection mm	Difference %
TB 10	2	Without	183	Control for Φ25mm	432	Control for Φ 25mm	17.35	Control for Φ 25mm
TB 18	2	4	244	33.3	518	19.9	22.5	29.7
TB 17	2	5	263	43.7	565	30.8	27.15	56.5
TB 19	0	Without	111	Control for Φ 16mm	111	Control for Φ 16mm	7.85	Control for Φ 16mm
TB 20	1	Without	123	10.8	281	153.1	13.03	66
TB 8	2	Without	205	84.7	446	301.8	18.49	135.5

4-3-3-12-2 Comparison between steel fiber and stirrups

Steel fiber is more effective than steel stirrups in shear strength by several times (Figs. 4-22 and 4-24) and Table (4-16) this due to the steel fiber improves the tensile strength of the section, and working as bridges after initiating of cracks between the cracks.

4-3-4 Cracking patterns for tested tapered-beams

Generally, at low loading, tapered-beams were free of cracks' forming. So, all tested tapered-beams' elastic were behaved. When load increased tensile stress increased and when exceeding concrete's tensile strength, cracks began to appear and the first crack generated at tapered-beam's bottom near support. New cracks were spread as loading level increased. The major diagonal cracks were beginning at support then toward loading' point. After the diagonal crack's propagation, load was increased until suddenly collapse occurred. For tapered-beam TB 13 in addition of the cracks mentioned above, its failure had inclined crack and initiated in shear span extended from the two concentrated loads position toward the bottom edges of NS CFRP bars of tapered-beam in the two sides due to the high confinement in the supports direction that provided by the orientation of NS CFRP bar, also flexural cracks were propagated. The cracks' rate was increases in the next cases: opening's presence, increased a/d ratio, and TB 13 because its crack failure difference from other tapered-beams. Cracks were reduced when: steel fibers ratio increased, and tensile bars distributed by two rows.

The crack patterns of nineteen tested tapered-beams until failure are in (Figs. 4-33 to 4-51).

The utilizing of NS CFRP bar with 30˚, 45˚ orientations in shear zone leads to increase in tapered-beam's shear capacity and deflection. The same goes when utilizing CFRP strip with 45˚ orientation.

It's clearly that tapered-beam with opening in the prismatic zone H1 cracked in lower load than tapered-beam with solid prismatic zone H1. This indicates that concrete core in tapered-beam with solid prismatic zone participates in apparent of cracks and under which load they occur.

Fig. 4-33 Crack pattern at failure of TB 1.

Fig. 4-34 Crack pattern at failure of TB 2.

Fig. 4-35 Crack pattern at failure of TB 3.

Fig. 4-36 Crack pattern at failure of TB 5.

First crack load = 187 kN
Failure load = 486 kN

Fig. 4-37 Crack pattern at failure of TB 6.

First crack load = 162 kN
Failure load = 370 kN

Fig. 4-38 Crack pattern at failure of TB 7.

First crack load = 205 kN
Failure load = 446 kN

Fig. 4-39 Crack pattern at failure of TB 8.

First crack load = 186 kN
Failure load = 460 kN

Fig. 4-40 Crack pattern at failure of TB 9.

Fig. 4-41 Crack pattern at failure of TB 10.

Fig. 4-42 Crack pattern at failure of TB 11.

Fig. 4-43 Crack pattern at failure of TB 12.

Fig. 4-44 Crack pattern at failure of TB 13.

Fig. 4-45 Crack pattern at failure of TB 14.

Fig. 4-46 Crack pattern at failure of TB 15.

Fig. 4-47 Crack pattern at failure of TB 16.

Fig. 4-48 Crack pattern at failure of TB 17.

Fig. 4-49 Crack pattern at failure of TB 18.

Fig. 4-50 Crack pattern at failure of TB 19.

Fig. 4-51 Crack pattern at failure of TB 20.

4-3-5 Shear failure in relation with angle of failure

In general, the tapered-beam's shear resistance increases with the failure angle's decreases for each group; this may due to the increasing of failure's path length (increasing the diagonal crack's length), and this could be observed in (Figs. 4-52 to 4-59), and (Table 4-17) elucidated failure load, angle of failure, and type of failure. When adding the inclination angle for each tapered-beam with its failure angle (that calculated with the horizontally), the failure angle will ranges from 31.197° to 36.297° this is for tapered-beams without stirrups and without CFRP bars/strips. As for tapered-beams those had NS CFRP bars, CFRP strips, and stirrups, the range of failure angle increases to become from 41.197° to 52.797° (Table 4-17).

Table 4-17 Angle of failure for tested tapered-beams.

Beam's ID	Ultimate Load kN	Failure Angle	Inclination angle	Total failure's angle (°)	Mode of failure
TB 1	416	20.4°	15.897°	36.297	Shear
TB 2	362	22.3°	9.697°	31.997	Shear
TB 3	375	21.2°	12.835°	34.035	Shear
TB 5	462	18.3°	15.897°	34.197	Shear
TB 6	486	17.1°	15.897°	32.997	Shear
TB 7	370	20.1°	15.897°	35.997	Shear
TB 8	446	16.9°	15.897°	32.797	Shear
TB 9	460	19.1°	15.897°	34.997	Shear
TB 10	432	19.8°	15.897°	35.697	Shear
TB 11	481	36.9°	15.897°	52.797	Shear
TB 12	590	30.1°	15.897°	45.997	Shear
TB 13	585	30.3°	15.897°	46.197	Shear
TB 14	441	29.3°	15.897°	45.197	Shear
TB 15	487	24.3°	15.897°	40.197	Shear
TB 16	465	28.6°	15.897°	44.497	Shear
TB 17	565	25.3°	15.897°	41.197	Shear
TB 18	518	27.9°	15.897°	43.797	Shear
TB 19	111	19.8°	15.897°	35.697	Shear
TB 20	281	17.2°	15.897°	33.097	Shear

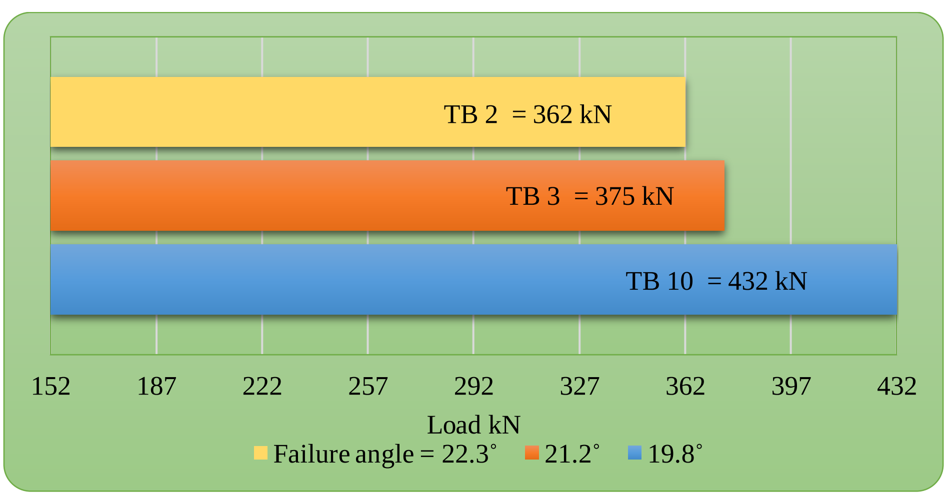


Fig. 4-52 Shear failure versus failure's angle for first group.

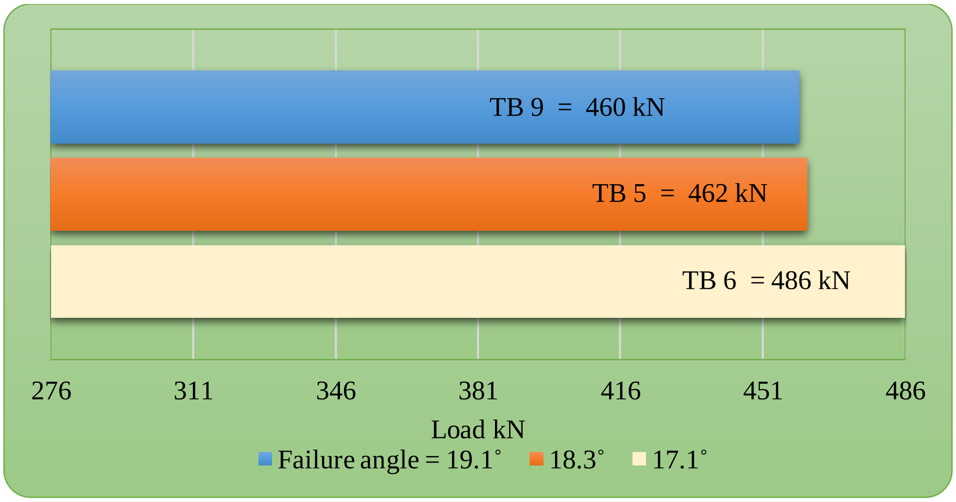


Fig. 4-53 Shear failure versus failure’s angle for second group.

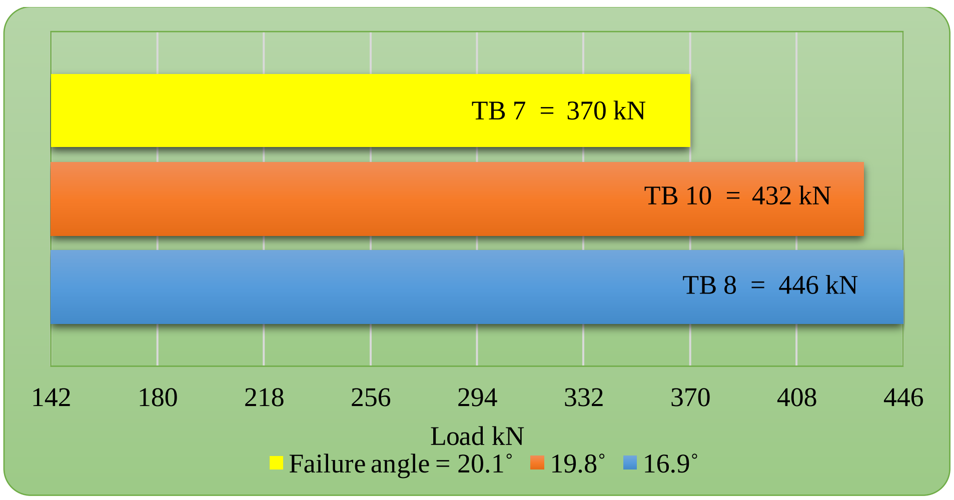


Fig. 4-54 Shear failure versus failure’s angle for third group.

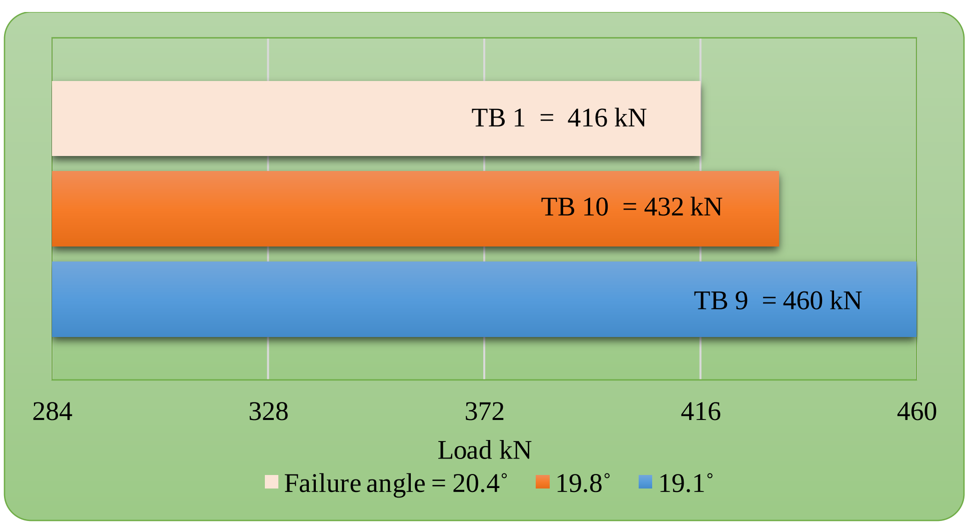


Fig. 4-55 Shear failure versus failure’s angle for fourth group.

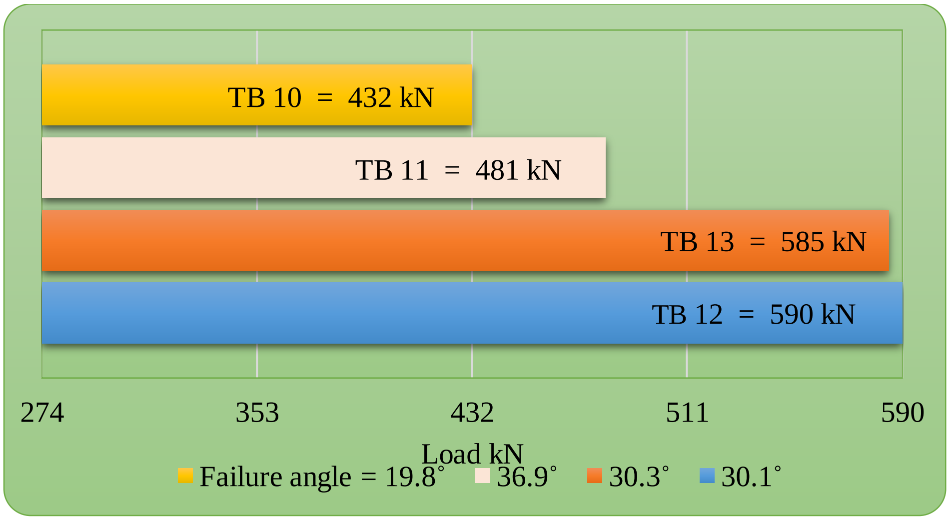


Fig. 4-56 Shear failure versus failure's angle for fifth group.

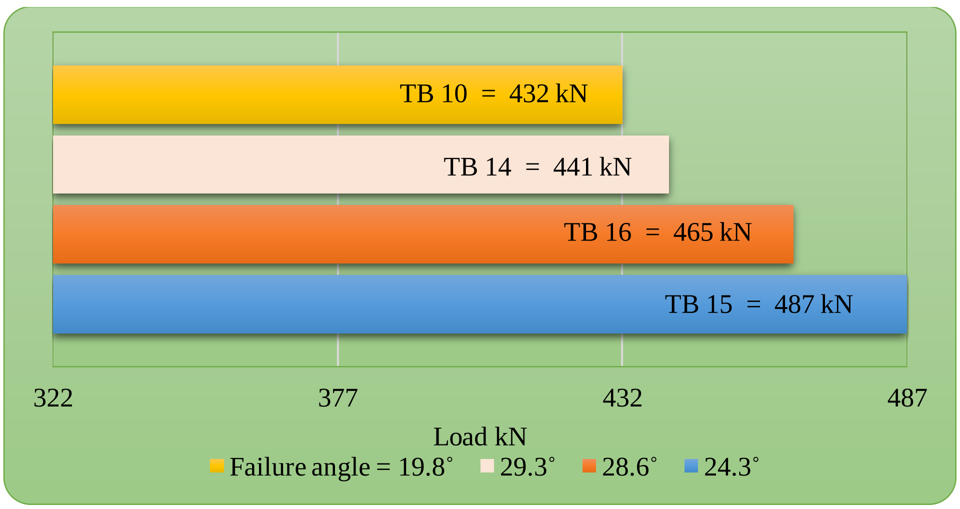


Fig. 4-57 Shear failure versus failure's angle for sixth group.

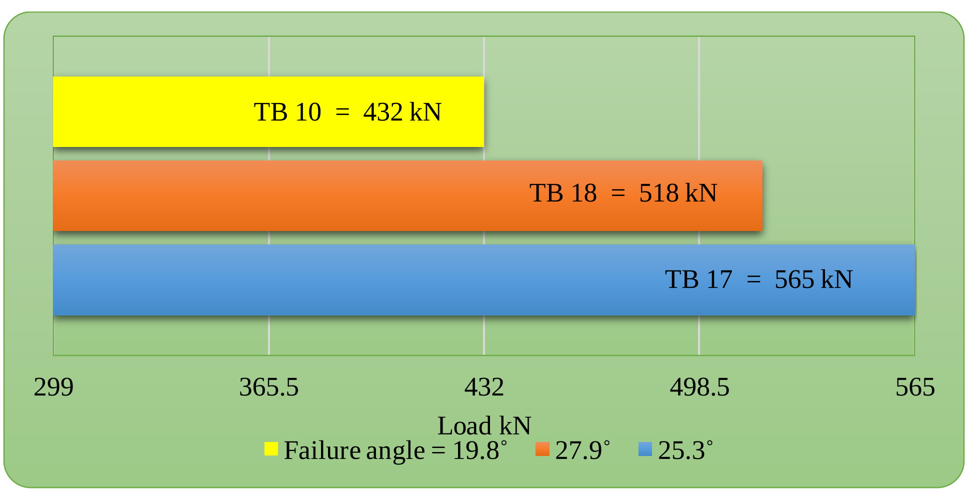


Fig. 4-58 Shear failure versus failure's angle for seventh group.

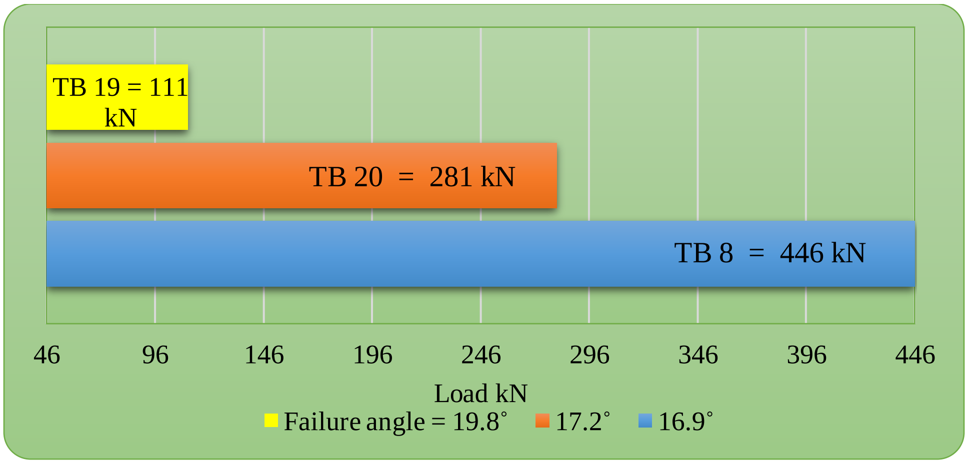


Fig. 4-59 Shear failure versus failure's angle for eighth group.

4-3-6 Designing methods

The theoretical shear failure loads were calculated by EXCEL sheets, appendix A. All tapered-beams as were mentioned in (Table 4-17), those were numbered with TB 1 to TB 16 and TB 19 and TB 20 were without stirrups, while the tapered-beams numbered by TB 17 and TB 18 were reinforced with 5 Φ 8mm and 4 Φ 8mm steel stirrups respectively. Tapered-beams those numbered TB 11 to TB 13 were strengthen with 4 Φ 6 mm of NS CFRP bars. The tapered-beams those numbered TB 14 to TB 16 were strengthen with 50 mm width of 4 CFRP strips. All tapered-beams were mentioned designed by Nasser's formulas [24], with utilizing (Albegmprli et al. 2018) formula [55] to calculate the dowel action contribution in shear capacity, all tapered-beams were designed the three methods to study which method from them is optimal for designing of UHPC tapered-beam. Also, had been utilized (ACI 440) [67] with Nasser's formulas [24] and Albegmprli et al. formula [55] for designed the tapered-beams with CFRP strips, and utilized (De Lorenzis and Nanni) [80] with (Nasser's method and Albegmprli et al. formula) for designed the tapered-beams with NS CFRP bars. Over and above, to ensure shear failure occurring, the maximum load of shear and flexural failure were chosen. Thus, the longitudinal reinforcement ratio is considered. Eventually, the shear failure type was also mentioned in Appendix A.

● *Irregular shape beam method*

The value of maximum experimental failure load was 1.49 of theoretical failure load, but experimental mode's failure was shear and theoretical was flexural Table (4-18), with same steel's area, this may due to material proportions, and diversified testing conditions.

Table (4-18) Theoretical and experimental results.

Beam ID	Theoretical results for three methods						Experimental results		$V_{Theo.}/V_{Exp}$ % For Nasser's formulas.
	Nasser		Deep beam		Irregular shape				
	Ultimate load kN	Failure Mode	Ultimate load kN	Failure Mode	Ultimate load kN	Failure Mode	Ultimate load kN	Failure Mode	
TB1	413.1	Shear	346	Flexural	348	Flexural	416	Shear	99
TB2	361.2	Shear	298	Flexural	275	Flexural	362	Shear	99
TB3	369.4	Shear	328	Flexural	322	Flexural	375	Shear	98
TB5	413.1	Shear	372	Flexural	370	Flexural	462	Shear	89
TB6	419.5	Shear	373	Flexural	371	Flexural	486	Shear	86
TB7	360.8	Shear	242	Flexural	232	Flexural	370	Shear	97
TB8	395.8	Shear	308	Flexural	329	Flexural	446	Shear	89
TB9	413	Shear	439	Flexural	445	Flexural	486	Shear	85
TB10	413.1	Shear	372	Flexural	370	Flexural	432	Shear	95
TB11	467.2	Shear	372	Flexural	370	Flexural	481	Shear	97
TB12	467.3	Shear	372	Flexural	370	Flexural	590	Shear	79
TB13	473.9	Shear	372	Flexural	370	Flexural	585	Shear	81
TB14	334.8	Shear	372	Flexural	370	Flexural	441	Shear	76
TB15	401.4	Shear	372	Flexural	370	Flexural	487	Shear	82
TB16	372.1	Shear	372	Flexural	370	Flexural	465	Shear	80
TB17	533.5	Shear	370	Flexural	370	Flexural	565	Shear	94
TB18	499.2	Shear	370	Flexural	370	Flexural	518	Shear	96
TB19	111.9	Shear	308	Flexural	378	Flexural	111	Shear	100
TB20	352	Shear	308	Flexural	329	Flexural	281	Shear	125

● *Deep beam method*

The value of maximum experimental failure load was 1.47 of theoretical failure load, but experimental mode's failure was shear and theoretical was flexural Table (4-18), with same steel's area, this may due to material proportions, widely different in geometries, as well as diversified testing conditions.

● *Nasser's method 2016 with Albegmprli et al. 2018 formula*

For UHPC tapered-beams with and without stirrups, mean value of theoretical to experimental shear failure load was 93.3%. For tapered-beam with 1% steel fiber the theoretical to experimental shear failure load was 125% this may due to the low concentration of steel fibers and may due to a lack of knowledge of the residual tensile stress in this low content. For tapered-beams have CFRP strips/bars, mean value of the theoretical to experimental shear failure load was 82.5%. Table (4-18), this difference may due to the deduction coefficients of CFRP.

CHAPTER FIVE

CONCLUSIONS

5-1 CONCLUSIONS

As result of the investigation, the essential conclusions could be extracted as following:

A- Material properties: -

1- UHPC mixtures are possible to develop from available locally materials, from three types of sand: sand #2, sand #3, and sand #4 with utilizing heat curing one week at 80°C and other three weeks inside water with room's temperature. The utilized of finer sand leads to increase the compressive strength, even though w/c ratio utilized with it is higher. The utilizing finer sand led to increase the water demand;

2- The compressive strength is affected by specimens' size. The cylindrical specimens' compressive strength was about 11% lesser than the cube specimens' compressive strength were casted with same mixture, and;

3- Utilizing of sand#5 super extra fine gradation (0.08 mm - 0.15 mm) leads to decrease compressive strength.

B- Tapered-beams: -

1- General phenomenon for tapered-beams' failures was that brittle collapses occur immediately after critical diagonal shear crack formation, then they can't be resisting more loading. Critical shear crack was initiated at former shear crack’s tips of or newly was formed in tapered-beam’s web and especially places near supports, that crack's type had called purely shear cracks or web shear cracks;

2- The presence of opening in prismatic (support) zone of tapered-beams contributed in decreasing of load carrying capacity, first crack load, deflections, and service deflection by 5.2%, 18.2%, 13.5%, and 19%

respectively. Openings' present within shear span caused considerable reduction in the shear capacity of tapered-beams.

3- The tapered-beam with one opening in the prismatic region has the same shear capacity and deflection of the tapered-beam with two openings. So the possibility of using two logical holes in each prismatic and non-prismatic region;

4- The shear capacity of tapered-beam increases by (19.9%) When stirrups number increased from (0 to 4 stirrups), and the first cracking load, service deflection, and deflection increased by (33.3%, 275%, and 29.7%) respectively;

5- The shear capacity of tapered-beam increased by (30.8%) when stirrups number increased from (0 to 5 stirrups), and first cracking load increased by (43.7%), service deflection increased by (240%), and deflection increased by (56.5%). Increasing stirrups amount will be increasing ultimate load-capacity and restraining diagonal cracks;

6- Shear capacity of tapered-beam increases by (11.3%, 35.4 %, 36.6 %) when utilized NS CFRP bar with orientations (0˚, 30°, 45°) respectively, and the first cracking load increased by (3.3%, 43.7%, and 57.4%) respectively, service deflection increases by (210%, 222%, and 225%) respectively, and deflection increased by (11.5%, 85.6%, and 99%) respectively, it's verified that inclined NS CFRP bar are more efficient than vertical ones, utilizing of CFRP leads to increasing of tapered-beam's shear resistance;

7- The shear capacity of tapered-beam increases by (2%, 7.6%, and 12.7%) when utilized CFRP strip with orientation (0˚, 30˚, and 45°) respectively, for CFRP strip 45˚ orientation first cracking load, service deflection, and deflection increased by (12%, 29.7%, and 2.6%) respectively, it's verified that inclined CFRP strip are more efficient than vertical ones;

8- Shear capacity of tapered-beams by NS CFRP bar is more aptitude than CFRP strip (in all orientations) in increasing of ultimate load capacity, first cracking load, service deflection, and final deflection by (21.1%, 40.5%, 73.3%, and 93.8%) respectively for 45˚ orientation;

9- The shear capacity of NS CFRP deformed bar 45˚ orientation is more efficiently from stirrups in increasing ultimate load, first cracking load, and deflection by (13.9%, 18%, and 53.4%) respectively, but decreasing in service deflection by (18.4%) comparing with same number of rods; instead of the stirrups' diameter was (8 mm) and NS CFRP deformed bar's was (6 mm) and this is also a point in favor of NS bar;

10- The utilizing of NS CFRP bars with 30˚ orientation led to keep the shear failure away from the support to the lower ends of the NS CFRP bars from beam's sides, it also led to the emergence of flexural cracks.

11- The shear capacity of tapered-beam with stirrups is more efficiently from CFRP strip 45˚ orientation for ultimate load, service-load, service deflection, and deflection by (6.3%, 19%, 212%, and 26.3%) respectively;

12- The increasing of tensile steel ratio led to promoted the tapered-beam's shear capacity, especially if longitudinal reinforcement distributed in more than one row, when steel's area varied from (981.7mm^2 in one row 1.79%) to (804.2mm^2 in two rows 1.57%) shear capacity increased by (3.2%) despite of steel's area was lesser by (26%), but distributed by two rows, also the first cracking load increased by (12%), service-load deflection increased by (90.2%), and the deflection increased by (6.6%) due to dowel action effects, when tensile bars distributed by two rows instead of one row even when utilized lesser area in case of two rows, this means the dowel action effect increase if tensile bar distributed by more one row with utilized big bars.

13- When steel's area increased from (628.3mm² in two rows 1.22%) to (981.7mm² in one row) shear capacity increased by (16.7%), also the first cracking load and the deflection increased by (12.9%, and 32.3%) respectively this decreasing due to utilized small bars (12 mm) in the second row, when tensile bar's ratio increased from (1.22% to 1.57%) when tensile bars were distributed by two rows, the ultimate load, first crack load, service load deflection, and deflection increased by (20.5%, 26.5%, 27%, and 41%) respectively;

14- Ultimate loads of UHPC tapered-beams decreased with a/d ratio increases; when a/d decrease from (2.94) to (2.3) led to increasing of failure loads and deflection about (10.6%, and 50.4%) respectively, but decreasing in first crack load, and service deflection by (7.9%, and 10%) respectively.

15- It's clearly from the test results that inclinations have a powerful impact on shear behavior as well as shear-capacity of concrete tapered-beam hasn't stirrups, which means inclined angle induces comparatively positive impacts to increase tapered-beams' shear strength, in other words; shear strength increases as inclined angle increases, when angle varied from (9.7°to15.9°) the failure load, first crack load, and deflection were increased by (19.3%, 24.5%, and 86.5%) respectively, but service deflection decreased by (47.3%);

16- The cracks' rate was increase in subsequent cases: opening's presence, increased a/d ratio, utilized of NS CFRP bars with 30˚ orientation. Cracks were reduced when steel fibers increased, tensile' bars distributed in two rows;

17- Steel fibers 2% are increasing shear capacity by 300 % which means four times, and more effective than steel stirrups, NS CFRP bars, and CFRP strips in shear's resistance, the steel fiber in UHPC tapered-beam is a function key role to restrain shear crack, steel fiber's presences leads to significant increasing in load's value at which first crack appeared 84.7%, and increased the deflection by 235% more than two times. When steel fibers didn't existence suddenly failure happened without warning (failure happened in same time crack's initiating);

18- When adding the failure angle for each UHPC tapered-beam with its inclination angle, the failure angle ranges from 31.197° to 36.297° for tapered-beams without stirrups and without CFRP bars/strips. As for tapered-beams those were with NS CFRP bars, CFRP strips, and stirrups the range of failure angle increases to become from 41.197° to 52.797°;

19- The deep beam and irregular section methods aren't suitable for designing of UHPC tapered-beam; because the experimental failure modes were shear and the theoretical failure modes were flexural, and the value of the experimental shear failure were exceed the value of the theoretical flexural was about 150%.

20- The nominal moment and shear resistant were computed by Naser's formulas with Albegmprli et al. 2018 formula to calculate dowel action contribution and compared with experimental results it was the better method for designing this beam's type, with conformity ratio of (93.3%) for UHPC tapered-beams with and without stirrups. For tapered-beam with 1% steel fiber the theoretical to experimental shear failure load was 125% this may due to the low concentration of steel fibers and may due to a lack of knowledge of the residual tensile stress in this low content. (82.5%) for tapered-beams have CFRP strips/bars, this difference may due to CFRP's deduction coefficients. And this method considered the ideal method for designing of UHPC tapered-beam.

5-2 RECOMMENDATIONS FOR FUTURE WORKS

A few points were proposed to enfold the recommended future work for UHPC tapered-beams:

- Study flexural behavior of UHPC tapered-beam.
- Research is recommended to study behavior of UHPC tapered-beam and comparing with UHPC prismatic-beam.
- Research is recommended to study shear-behavior of UHPC tapered-beam with different hollows shapes and direction.

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APPENDIX A

Nasser's formulas

			TAPERED BEAM (1) 2 φ 25 H = 405mm (W/O) Stirrups	φ of stirrups (mm) =	8
Effective Depth (mm) for shear for tapered beam	(ds) =	216.225		θ	15.897
Total thickness	H (Total)	405		Tan (θ)	0.2848
Compressive Strength (Mpa)	fc' =	135		Effective Depth (at mid span) (mm) d	364.5
Beam Width (mm)	b =	150		Yield Stress of Steel main Reinf.(Mpa) fy	420
Beam's depth at support (mm)	H 1 =	180		Modulus of Elasticity of Steel (Mpa) Es	200000
Concrete Modulus of Elasticity (Mpa)	Ec =	50000		Ultimate Concrete Strain εcu	0.0035
Ratio of (α1) to (ɣ)	α =	0.65		Overhange length (m)	0.16
Ratio of Average concrete Stress to (fc')	α1 =	0.85		Load gap (m)	0.20
Concrete cover (mm)	c =	20		Shear Span (m)	0.690
Daimeter of main Reinforcemdnt (mm)	φ =	25		Shear Safety Factor	1.3
Span Length (m)	L =	1.58		Concrete Desity (kN/m3)	25
		Failure's angle	35	H2 =	225
Use ((2 φ 25) (Ast) = 981.71875	mm²	Ln/2 =	790	Length of Beam L	1.9
				Quantity	0.0779625

A =	α.fc'.b -0.5*(α.fc')^2 .b/(Ec* εcu) + 0.4*√fc'.b	10559.61022	c =	129.69877	β =	0.65
B =	- 0.4*√fc'.b.H + As.Es.εcu	561718.4646	a =	84.304201	hx =	h * (1 + x / 2L)
D =	- As.Es.εcu.d	-250485539.1		I =		(bh^3 / 12) * (1+ x / 2 L)^3
	C = [α.fc'.b -0.5*(α.fc')^2 .b/(Ec* εcu)]* c^2 =				165.904	kN.m
	T = 0.4*√fc'.b.H * c - 0.4*√fc'.b.*c^2 - As.Es.εcu * c + As.Es.εcu.d =				165.904	kN.m

				dc=ds*F	
c =	129.699	r = (εcu-α.fc'/Ec)*(c / εcu) =	64.66410117	ds=180-(20+8+25/2) =	139.5
C1 =	α.fc' * b * r / 10^6 =		0.851141232	F=(1-3.04 tan)^(-0.608) <= 1.55 =	3.38383
C2 =	0.5*α.fc' * (b * (c - r) -π 25^2) / 10^6 =		0.341861055	Use F= 1.55	1.55
T1 =	0.4*√fc' * (b * (h-c) - π 25^2) / 10^6 =		0.025941346	dc =	216.225
T2 =	(As.Es.εcu.(d / c - 1)) =		1.244083799		

Calculation of Tensile strain(εf) & Stress (fpf) of Steel bar

εs = εcu *(d-c) / c =	0.006336254	fs =	1267.250726	> fy =	420	Not OK
					Use fy =	420

Calculation of Flextural Strength (Mn) : -

			Inclination angle of reinforcement = ζ	12.282849	
			φ 8 (As2) = л ((8^2) / 4)) * 2	100.53088	
			Use fy for stirrups =	420	
Mn1 = C1 * (c - r / 2)	82.8728				
Mn2 = C2 * 2/3*(c - r)	14.8219	Vc =	(0.18*√fc'* (b*ds - (3.141592654*25^2)) / k11) / 1000		49.0199
Mn3 = T1 * (h - c)/2	0.65244	Vf =	((0.4*(√fc')*0.9* (b* ds - π 25^2) / k11) /(TAN(35*3.14159/180))) /1000		140.015
Mn4 = As *fs *(d-c)	96.8137	Vs =	(0.9 *(As2 /800)* fy * ds /1.3) /1000)		0
	195.161	Dowel action Vda	0.2*As*fy*sinζ		17.5433
					206.579

M D.L of Beam Self Weight	((b * H - 50 * 50)*G12 / 10^6 + (0.5 * L* 120 *b - ((L + 840)/2) *50*50)) / 10^9) *k12 * L^2 / 8) / 2)		0.1469938		
		M total =	195.01384		
	M total = (Pu,f * shear span) / 2	Pu,f = (2 * M) /a	565.25751	Vt =	413.1571184
		Theoritecal Failure's load =			413.1571184
		Experemental failure's load =			416
		Accuracy of the Method % =			0.99316615
Pu,f > Vt	Shear Domiant				

TAPERED BEAM (2) (W/O) Stirrups

			φ of stirrups (mm)	*8*
Effective Depth (mm) for shear for tapered beam	*(ds) =*	*198*	*θ*	*9.697*
Total thickness	*H (Total)*	*315*	*Tan (θ)*	*0.170879*
Compressive Strength (Mpa)	*fc' =*	*135*	*Effective Depth (at mid span) (mm) d*	*274.5*
Beam Width (mm)	*b =*	*150*	*Yield Stress of Steel -main Reinf.(Mpa) fy*	*420*
Beam's depth at support (mm)	*H 1 =*	*180*	*Modulus of Elasticity of Steel (Mpa) Es*	*200000*
Concrete Modulus of Elasticity (Mpa)	*Ec =*	*50000*	*Ultimate Concrete Strain εcu*	*0.0035*
Ratio of (α1) to (ɣ)	*α =*	*0.65*	*Overhange length (m)*	*0.16*
Ratio of Average concrete Stress to (fc')	*α1 =*	*0.85*	*Load gap (m)*	*0.30*
Concrete cover (mm)	*c =*	*20*	*Shear Span (m)*	*0.640*
Daimeter of main Reinforcemdnt (mm)	*φ =*	*25*	*Shear Safety Factor*	*1.3*
Span Length (m)	*L =*	*1.58*	*Concrete Desity (kN/m3)*	*25*

			Failure's angle	*35*	*H2 =*	*135*	
Use ((2 φ 25) (Ast) =	*981.71875*	*mm²*	*Ln/2 =*	*790*	*Length of Beam L =*		*1.9*
A =	*α.fc'.b -0.5*(α.fc')^2 .b/(Ec* εcu) + 0.4*√fc'.b*	*=*	*10559.61022*	*c =*	*109.6796554*	*β =*	*0.65*
B =	*- 0.4*√fc'.b.H + As.Es.εcu*	*=*	*561718.4646*	*a =*	*71.29177602*		
D =	*- As.Es.εcu.d*	*=*	*-188637257.8*				
		I =	*(bh^3 / 12) * (1+ x / 2 L)^3*				

C = [α.fc'.b -0.5(α.fc')^2 .b/(Ec* εcu)]* c^2*	*118.6418722*	*kN.m*
T = 0.4√fc'.b.H * c - 0.4*√fc'.b.*c^2 - As.Es.εcu * c + As.Es.εcu.d*	*118.6418722*	*kN.m*

c =	*109.68*	*r = (εcu-α.fc'/Ec)*(c / εcu) =*	*54.68314248*
C1 =	*α.fc' * b * r / 10^6 =*		*0.719766863*
C2 =	*0.5*α.fc' * (b * (c - r) -π 25^2) / 10^6 =*		*0.27579744*
T1 =	*0.4*√fc' * (b * (h-c) - π 25^2) / 10^6 =*		*0.039897412*
T2 =	*(As.Es.εcu.(d / c - 1)) =*		*1.032689749*

Calculation of Tensile strain(εf) & Stress (fpf) of Steel bar

*εs = εcu *(d-c) / c =*	*0.005259601*	*fs =*	*1051.920165*	*> fy =*	*420*	*Not OK*
				Use fy =	*420*	

Calculation of Flextural Strength (Mn) : -

Inclination angle of reinforcement = ζ	*5.92594083*
*φ 8 (As2) = л ((8^2) / 4)) * 2*	*100.53088*
Use fy for stirrups =	*420*

Mn1	*C1 * (c - r / 2)*	*59.2642*			
Mn2	*C2 * 2/3*(c - r)*	*10.1119*	*Vc =*	*(0.18*√fc'* (b*ds - (3.141592654*25^2)) / k11) / 1000*	*44.62187*
Mn3	*T1 * (h - c)/2*	*1.4028*	*Vf =*	*((0.4*(√fc')*0.9* (b* ds - π 25^2) / k11) /(TAN(35*3.14159/180))) /1000*	*127.453417*
Mn4	*As *fs *(d-c)*	*67.959*	*Vs =*	*(0.9 *(As2 /800)* fy * ds /1.3) /1000)*	*0*
		138.738	*Dowel action Vda*	*0.2*As*fy*sinθ*	*8.513852467*
					180.5891394

M D.L of Beam Self Weight	*((b * H - 50 * 50)*G12 / 10^6 + (0.5 * L* 120 *b - ((L + 840)/2) *50*50)) / 10^9) *k12 * L^2 / 8) / 2)*	*0.14695223*
	M total =	*138.591037*

*M total = (Pu,f * shear span) / 2*	*Pu,f = (2 * M) /a*	*433.096992*	*Vt =*	*361.1782789*
	Theoritecal Failure's load =			*361.1782789*
	Experemental failure's load =			*362*
Pu,f > Vt	*Shear Domiant*	*Accuracy of the Method =*		*0.997730052*

						TAPERED BEAM (3) (W/O) Stirrups	φ of stirrups (mm)		8
	Effective Depth (mm) for shear for tapered beam			(ds) =	197.5		θ		12.835
	Total thickness			H (Total)	360		Tan (θ)		0.2278369
	Compressive Strength (Mpa)			fc' =	135		Effective Depth (at mid span) (mm) d		319.5
	Beam Width (mm)			b =	150		Yield Stress of Steel -main Reinf.(Mpa) fy		420
	Beam's depth at support (mm)			H 1 =	180		Modulus of Elasticity of Steel (Mpa) Es		200000
	Concrete Modulus of Elasticity (Mpa)			Ec =	50000		Ultimate Concrete Strain εcu		0.0035
	Ratio of (α1) to (ɣ)			α =	0.65		Overhange length (m)		0.16
	Ratio of Average concrete Stress to (fc')			α1 =	0.85		Load gap (m)		0.30
	Concrete cover (mm)			c =	20		Shear Span (m)		0.640
	Daimeter of main Reinforcemdnt (mm)			φ =	25		Shear Safety Factor		1.3
	Span Length (m)			L =	1.58		Concrete Desity (kN/m3)		25
					Failure's angle	35	H2 =	180	
Use ((2 φ 25) (Ast) =			981.71875	mm²	Ln/2 =	790	Length of Beam L =		1.9
A =	α.fc'.b -0.5*(α.fc')^2 .b/(Ec* εcu) + 0.4*√fc'.b =				10559.61022	c =	120.0312611	β =	0.65
B =	- 0.4*√fc'.b.H + As.Es.εcu =				561718.4646	a =	78.02031974		
D =	- As.Es.εcu.d =				-219561398.4				
				I =	(bh^3 / 12) * (1+ x / 2 L)^3				
	C = [α.fc'.b -0.5*(α.fc')^2 .b/(Ec* εcu)]* c^2				142.0936188	kN.m			
	T = 0.4*√fc'.b.H * c - 0.4*√fc'.b.*c^2 - As.Es.εcu * c + As.Es.εcu.d				142.0936188	kN.m			
c =		120.0312611	r = (εcu-α.fc'/Ec)*(c / εcu) =		59.84415734				
C1 =	α.fc' * b * r / 10^6 =				0.787698721				
C2 =	0.5*α.fc' * (b * (c - r) -π 25^2) / 10^6 =				0.309958016				
T1 =	0.4*√fc' * (b * (h-c) - π 25^2) / 10^6 =				0.032680925				
T2 =	(As.Es.εcu.(d / c - 1)) =				1.141998671				

Calculation of Tensile strain(εf) & Stress (fpf) of Steel bar

εs = εcu *(d-c) / c =		0.005816323	fs =	1163.264602	> fy =	420	Not OK
					Use fy =	420	

Calculation of Flextural Strength (Mn) : -

				Inclination angle of reinforcement = ζ		9.132709185		
				φ 8 (As2) = л ((8^2) / 4)) * 2		100.53088		
Mn1	C1 * (c - r / 2)	70.97888777		Use fy for stirrups =		420		
Mn2	C2 * 2/3*(c - r)	12.43698351	Vc =	(0.18*√fc'* (b*ds - (3.141592654*25^2)) / k12) / 1000				44.50121167
Mn3	T1 * (h - c)/2	0.979916923	Vf =	((0.4*(√fc')*0.9* (b* ds - π 25^2) / k12) /(TAN(35*3.14159/180))) /1000				127.1087807
Mn4	As *fs *(d-c)	82.24532441	Vs =	(0.9 *(As2 /800)* fy * ds /1.3) /1000)				0
		166.6411126	Dowel action Vda	0.2*As*fy*sinθ				13.08887806
								184.6988704
M D.L of Beam Self Weight	((b * H - 50 * 50)*G12 / 10^6 + (0.5 * L* 120 *b - ((L + 840)/2) *50*50)) / 10^9) *k13 * L^2 / 8) / 2)					0.146973034		
					M total =	166.4941396		
			M total = (Pu,f * shear span) / 2		Pu,f = (2 * M) /a	520.2941862	Vt =	369.3977409
					Theoritecal Failure's load =			369.3977409
					Experemental failure's load =			375
			Pu,f > Vt	Shear Domiant	Accuracy of the Method =			0.985060642

Nasser's formulas

			TAPERED BEAM (5) 2 φ 25 (W/O) Stirrups	φ of stirrups (mm) =	8
Effective Depth (mm) for shear for tapered beam	(ds) =	216.225		θ	15.897
Total thickness	H (Total)	405		Tan (θ)	0.2848
Compressive Strength (Mpa)	fc' =	135		Effective Depth (at mid span) (mm) d	364.5
Beam Width (mm)	b =	150		Yield Stress of Steel main Reinf.(Mpa) fy	420
Beam's depth at support (mm)	H 1 =	180		Modulus of Elasticity of Steel (Mpa) Es	200000
Concrete Modulus of Elasticity (Mpa)	Ec =	50000		Ultimate Concrete Strain εcu	0.0035
Ratio of (α1) to (ɣ)	α =	0.65		Overhange length (m)	0.16
Ratio of Average concrete Stress to (fc')	α1 =	0.85		Load gap (m)	0.20
Concrete cover (mm)	c =	20		Shear Span (m)	0.690
Daimeter of main Reinforcemdnt (mm)	φ =	25		Shear Safety Factor	1.3
Span Length (m)	L =	1.58		Concrete Desity (kN/m3)	25
		Failure's angle	35	H2 =	225
Use ((2 φ 25) (Ast) = 981.71875	mm²	Ln/2 =	790	Length of Beam L	1.9

					Quantity	0.0779625
A =	α.fc'.b -0.5*(α.fc')^2 .b/(Ec* εcu) + 0.4*√fc'.b	10559.61022	c =	129.699	β =	0.65
B =	- 0.4*√fc'.b.H + As.Es.εcu	561718.4646	a =	84.3042	hx =	h * (1 + x / 2L)
D =	- As.Es.εcu.d	-250485539.1		I =	(bh^3 / 12) * (1+ x / 2 L)^3	
	C = [α.fc'.b -0.5*(α.fc')^2 .b/(Ec* εcu)]* c^2 =				165.904266	kN.m
	T = 0.4*√fc'.b.H * c - 0.4*√fc'.b.*c^2 - As.Es.εcu * c + As.Es.εcu.d =				165.904266	kN.m

			dc=ds*F	
c = 129.699	r = (εcu-α.fc'/Ec)*(c / εcu) =	64.66410117	ds=180-(20+8+25/2) =	139.5
C1 =	α.fc' * b * r / 10^6 =	0.851141232	F=(1-3.04 tan)^(-0.608) <= 1.55 =	3.38383
C2 =	0.5*α.fc' * (b * (c - r) -π 25^2) / 10^6 =	0.341861055	Use F= 1.55	1.55
T1 =	0.4*√fc' * b * (h-c) / 10^6 =	0.035066849	dc =	216.225
T2 =	(As.Es.εcu.(d / c - 1)) =	1.244083799		

Calculation of Tensile strain(εf) & Stress (fpf) of Steel bar

εs = εcu *(d-c) / c =	0.006336254	fs =	1267.250726	> fy =	420	Not OK
					Use fy =	420

Calculation of Flextural Strength (Mn) : -

			Inclination angle of reinforcement = ζ	12.2828	
			φ 8 (As2) = л ((8^2) / 4)) * 2	100.531	
Mn1 =	C1 * (c - r / 2)	82.8728	Use fy for stirrups =	420	
Mn2 =	C2 * 2/3*(c - r)	14.8219	Vc =	(0.18*√fc'* (b*ds - (3.141592654*25^2)) / k11) / 1000	49.01986603
Mn3 =	T1 * (h - c)/2	0.88195	Vf =	((0.4*(√fc')*0.9* (b* ds - π 25^2) / k11) /(TAN(35*3.14159/180))) /1000	140.0154093
Mn4 =	As *fs *(d-c)	96.8137	Vs =	(0.9 *(As2 /800)* fy * ds /1.3) /1000)	0
		195.39	Dowel action Vda	0.2*As*fy*sinζ	17.54328389
					206.5785592

M D.L of Beam Self Weight	((b * H - 50 * 50)*G12 / 10^6 + (0.5 * L* 120 *b - ((L + 840)/2) *50*50)) / 10^9) *k12 * L^2 / 8) / 2)	0.14699			
		M total =	195.243		
M total = (Pu,f * shear span) / 2		Pu,f = (2 * M) /a	565.923	Vt =	413.1571184
		Theoritecal Failure's load =			413.1571184
		Experemental failure's load =			462
Pu,f > Vt	Shear Domiant	Accuracy of the Method % =			0.894279477

				TAPERED BEAM (6) 2 φ 25 (W/O) Stirrups	φ of stirrups (mm) =	8
	Effective Depth (mm) for shear for tapered beam	(ds) =	216.225		θ	15.897
	Total thickness	H (Total)	405		Tan (θ)	0.2848
	Compressive Strength (Mpa)	fc' =	135		Effective Depth (at mid span) (mm) d	364.5
	Beam Width (mm)	b =	150		Yield Stress of Steel main Reinf.(Mpa) fy	420
	Beam's depth at support (mm)	H 1 =	180		Modulus of Elasticity of Steel (Mpa) Es	200000
Nasser's formulas	Concrete Modulus of Elasticity (Mpa)	Ec =	50000		Ultimate Concrete Strain εcu	0.0035
	Ratio of (α1) to (ɣ)	α =	0.65		Overhange length (m)	0.16
	Ratio of Average concrete Stress to (fc')	α1 =	0.85		Load gap (m)	0.20
	Concrete cover (mm)	c =	20		Shear Span (m)	0.690
	Daimeter of main Reinforcemdnt (mm)	φ =	25		Shear Safety Factor	1.3
	Span Length (m)	L =	1.58		Concrete Desity (kN/m3)	25
			Failure's angle	35	H2 =	225
Use ((2 φ 25) (Ast) =	981.71875	mm²	Ln/2 =	790	Length of Beam L	1.9

					Quantity	0.0779625
A =	α.fc'.b -0.5*(α.fc')^2 .b/(Ec* εcu) + 0.4*√fc'.b	10559.61022	c =	129.6987703	β =	0.65
B =	- 0.4*√fc'.b.H + As.Es.εcu	561718.4646	a =	84.30420067	hx =	h * (1 + x / 2L)
D =	- As.Es.εcu.d	-250485539.1		I =	(bh^3 / 12) * (1+ x / 2 L)^3	
	C = [α.fc'.b -0.5*(α.fc')^2 .b/(Ec* εcu)]* c^2		=		165.904	kN.m
	T = 0.4*√fc'.b.H * c - 0.4*√fc'.b.*c^2 - As.Es.εcu * c + As.Es.εcu.d		=		165.904	kN.m

				dc=ds*F	
c =	129.699	r = (εcu-α.fc'/Ec)*(c / εcu) =	64.66410117	ds=180-(20+8+25/2) =	139.5
C1 =	α.fc' * b * r / 10^6	=	0.851141232	F=(1-3.04 tan)^(-0.608) <= 1.55 =	3.38383
C2 =	0.5*α.fc' * b * (c - r) / 10^6	=	0.428009416	Use F= 1.55	1.55
T1 =	0.4*√fc' * (b * (h-c) - π 25^2) / 10^6	=	0.025941346	dc =	216.225
T2 =	(As.Es.εcu.(d / c - 1))	=	1.244083799		

Calculation of Tensile strain(εf) & Stress (fpf) of Steel bar

εs = εcu *(d-c) / c =	0.006336254	fs =	1267.250726	> fy =	420	Not OK
					Use fy =	420

Calculation of Flextural Strength (Mn) : -

			Inclination angle of reinforcement = ζ	12.28284898	
			φ 8 (As2) = л ((8^2) / 4)) * 2	100.53088	
Mn1 = C1 * (c - r / 2)	82.8728		Use fy for stirrups =	420	
Mn2 = C2 * 2/3*(c - r)	18.557	Vc =	(0.18*√fc'* b*ds / k11) / 1000		52.1787
Mn3 = T1 * (h - c)/2	0.65244	Vf =	((0.4*(√fc')*0.9* (b* ds - π 25^2) / k11) /(TAN(35*3.14159/180))) /1000		140.015
Mn4 = As *fs *(d-c)	96.8137	Vs =	(0.9 *(As2 /800)* fy * ds /1.3) /1000)		0
	198.896	Dowel action Vda	0.2*As*fy*sinζ		17.5433
					209.737

M D.L of Beam Self Weight	((b * H - 50 * 50)*G12 / 10^6 + (0.5 * L* 120 *b - ((L + 840)/2) *50*50)) / 10^9) *k12 * L^2 / 8) / 2)	0.146993834		
	M total =	198.7489271		
M total = (Pu,f * shear span) / 2	Pu,f = (2 * M) /a	576.0838468	Vt =	419.4747737
	Theoritecal Failure's load =			419.4747737
	Experemental failure's load =			486
	Accuracy of the Method % =			0.863116818
Pu,f > Vt	Shear Domiant			

Nasser's formulas

			TAPERED BEAM (7) (W/O)Stirrups	φ of stirrups (mm) =	8
Effective Depth (mm) for shear for tapered beam	(ds) =	195.3		θ	15.897
Total thickness	H (Total)	405		Tan (θ)	0.2848
Compressive Strength (Mpa)	fc' =	135		Effective Depth (at mid span) (mm) d	346
Beam Width (mm)	b =	150		Yield Stress of main Reinf.(Mpa) fy	420
beam depth at support (mm)	H 1 =	180		Modulus of Elasticity of Steel Es	200000
Concrete Modulus of Elasticity (Mpa)	Ec =	50000		Ultimate Concrete Strain εcu	0.0035
Ratio of (α1) to (ɣ)	α =	0.65		Overhange length (m)	0.16
Ratio of Average concrete Stress to (fc')	α1 =	0.85		Load gap (m)	0.30
Concrete cover (mm)	c =	20		Shear Span (m)	0.640
Daimeter of main Reinforcemdnt (mm)	φ =	16		Shear Safety Factor	1.3
Span Length (m)	L =	1.58		Concrete Desity (kN/m3)	25

		Failure's angle	35	H2 =	225	Quantity	0.07796

Use ((2 φ 16 +2Φ12) (Ast) =	628.3	mm²	Ln/2 =	790	Length of Beam L =	1.9
A =	α.fc'.b -0.5*(α.fc')^2 .b/(Ec* εcu) + 0.4*√fc'.b	10559.61022	c =	106.0814	β =	0.65
B =	- 0.4*√fc'.b.H + As.Es.εcu	314325.3396	a =	68.9529		
D =	- As.Es.εcu.d	-152174260	hx =	h * (1 + x / 2L)		
	C = [α.fc'.b -0.5*(α.fc')^2 .b/(Ec* εcu)]* c^2 =			110.985	kN.m	
	T = 0.4*√fc'.b.H * c - 0.4*√fc'.b.*c^2 - As.Es.εcu * c + As.Es.εcu.d =			110.985	kN.m	

c =	106.081	r = (εcu-α.fc'/Ec)*(c / εcu) =	52.88917709	dc=ds*F	
C1 =	α.fc' * b * r / 10^6 =		0.696153794	ds=180-(20+8+16+15) =	126
C2 =	0.5*α.fc' * (b * (c - r) -π 25^2) / 10^6 =		0.263923246	F=(1-3.04 tan)^(-0.608) <= 1.55	3.38383
T1 =	0.4*√fc' * (b * (h-c) - π 25^2) / 10^6 =		0.042405859	Use F= 1.55	1.55
T2 =	(As.Es.εcu.(d / c - 1)) =		0.99469404	dc =	195.3

Calculation of Tensile strain(εf) & Stress (fpf) of Steel bar

εs = εcu *(d-c) / c =	0.007915757	fs =	1583.151424	> fy =	420	Not OK
				Use fy =	420	

Calculation of flextural strength(Mn) : -

Inclination angle of reinforcement = ζ	11.86673527
φ 8 (As2) = л ((8^2) / 4)) * 2	100.53088

Mn1	C1 * (c - r / 2)	55.4395	Use fy for stirrups =	420	
Mn2	C2 * 2/3*(c - r)	9.35912	Vc =	(0.18*√fc'* (b*ds - (3.141592654*25^2)) / M12) / 1000	43.970315
Mn3	T1 * (h - c)/2	1.56729	Vf =	((0.4*(√fc')*0.9* (b* ds - π 25^2) / M12) /(TAN(35*3.14159/180))) /1000	125.59238
Mn4	As *fs *(d-c)	63.3111	Vs =	(0.9 *(As2 /800)* fy * ds /1.3) /1000)	0
		129.677	Dowel action Vda =	0.2*As*fy*sinζ	10.847472
					180.41017

M D.L of Beam Self Weight	((b * H - 50 * 50)*G12 / 10^6 + (0.5 * L* 120 *b - ((L + 840)/2) *50*50)) / 10^9) *M13 * L^2 / 8) / 2)	0.14699		
	M total =	129.53		
M total = (Pu,f * shear span) / 2	Pu,f = (2 * M) /a	404.781	Vt =	360.82
	Theoritecal Failure's load =			360.82
	Experemental failure's load =			370
Pu,f > Vt	Shear Domiant	Accuracy of the Method =		0.97519

TAPERED BEAMS (8)
(W/O) Stirrups

Nasser's formulas		
Effective Depth (mm) for shear for tapered beam	(ds) =	210.8
Total thickness	H (Total)	405
Compressive Strength (Mpa)	fc' =	135
Beam Width (mm)	b =	150
beam depth at support (mm)	H 1 =	180
Concrete Modulus of Elasticity (Mpa)	Ec =	50000
Ratio of (α1) to (ɣ)	α =	0.65
Ratio of Average concrete Stress to (fc')	α1 =	0.85
Concrete cover (mm)	c =	20
Daimeter of main Reinforcemdnt (mm)	φ =	16
Span Length (m)	L =	1.58
		Failure's angle: 35

φ of stirrups (mm) =	8
θ	15.897
Tan (θ)	0.2848
Effective Depth (at mid span) (mm) d	346
Yield Stress of main Reinf.(Mpa) fy	420
Modulus of Elasticity of Steel Es	200000
Ultimate Concrete Strain εcu	0.0035
Overhange length (m)	0.16
Load gap (m)	0.30
Shear Span (m)	0.640
Shear Safety Factor	1.3
Concrete Desity (kN/m3)	25
H2 = 225	Quantity 0.07796
Length of Beam L =	1.9

Use ((4 φ 16) (Ast) =	804.224	mm²	Ln/2 =	790		
A =	α.fc'.b -0.5*(α.fc')^2 .b/(Ec* εcu) + 0.4*√fc'.b =	10559.61022	c =	116.6723909	β =	0.65
B =	- 0.4*√fc'.b.H + As.Es.εcu =	437472.1396	a =	75.83705409		
D =	- As.Es.εcu.d =	-194783052.8	hx =	h * (1 + x / 2L)		
	C = [α.fc'.b -0.5*(α.fc')^2 .b/(Ec* εcu)]* c^2 =			134.252392	kN.m	
	T = 0.4*√fc'.b.H * c - 0.4*√fc'.b.*c^2 - As.Es.εcu * c + As.Es.εcu.d =			134.252392	kN.m	

c =	116.6723909	r = (εcu-α.fc'/Ec)*(c / εcu) =	58.16952061	dc=ds*F	
C1 =	α.fc' * b * r / 10^6 =		0.765656315	ds=180-(20+8+16) =	136
C2 =	0.5*α.fc' * (b * (c - r) -π 25^2) / 10^6 =		0.298873654	F=(1-3.04 tan)^(-0.608) <= 1.55	= 3.383826421
T1 =	0.4*√fc' * (b * (h-c) - π 25^2) / 10^6 =		0.035022518	Use F= 1.55	1.55
T2 =	(As.Es.εcu.(d / c - 1)) =		1.106530311	dc =	210.8

Calculation of Tensile strain(εf) & Stress (fpf) of Steel bar

εs = εcu *(d-c) / c =	0.006879491	fs =	1375.898146	> fy =	420	Not OK
				Use fy =	420	

Calculation of Flextural Strength (Mn) : -

Inclination angle of reinforcement = ζ	11.86673527
φ 8 (As2) = л ((8^2) / 4)) * 2	100.53088
Use fy for stirrups =	420

Mn1	C1 * (c - r / 2)	67.0620225			
Mn2	C2 * 2/3*(c - r)	11.65664442	Vc =	(0.18*√fc'* (b*ds - (3.141592654*25^2)) / M11) / 1000	47.71072318
Mn3	T1 * (h - c)/2	1.10894615	Vf =	((0.4*(√fc')*0.9* (b* ds - π 25^2) / M11) /(TAN(35*3.14159/180))) /1000	136.2761057
Mn4	As *fs *(d-c)	77.46092218	Vs =	(0.9 *(As2 /800)* fy * ds /1.3) /1000)	0
		157.2885352	Dowel action Vda =	0.2*As*fy*sinζ	13.89169387
					197.8785227

M D.L of Beam Self Weight	((b * H - 50 * 50)*G12 / 10^6 + (0.5 * L* 120 *b - ((L + 840)/2) *50*50)) / 10^9) *M12 * L^2 / 8) / 2)	0.146993834			
		M total =	157.1415414		
M total = (Pu,f * shear span) / 2		Pu,f = (2 * M) /a	491.0673169	Vt =	395.7570455
		Theoritecal Failure's load =			395.7570455
		Experemental failure's load =			446
Pu,f > Vt	Shear Domiant	Accuracy of the Method =			0.887347636

Nasser's formulas

TAPERED BEAM (9) 2 φ 25 (W/O) Stirrups

			φ of stirrups (mm) =	8
Effective Depth (mm) for shear for tapered beam	(ds) =	216.225	θ	15.897
Total thickness	H (Total)	405	Tan (θ)	0.2848
Compressive Strength (Mpa)	fc' =	135	Effective Depth (at mid span) (mm) d	364.3
Beam Width (mm)	b =	150	Yield Stress of Steel main Reinf.(Mpa) fy	420
Beam's depth at support (mm)	H 1 =	180	Modulus of Elasticity of Steel (Mpa) Es	200000
Concrete Modulus of Elasticity (Mpa)	Ec =	50000	Ultimate Concrete Strain εcu	0.0035
Ratio of (α1) to (ɣ)	α =	0.65	Overhange length (m)	0.16
Ratio of Average concrete Stress to (fc')	α1 =	0.85	Load gap (m)	0.50
Concrete cover (mm)	c =	20	Shear Span (m)	0.540
Daimeter of main Reinforcemdnt (mm)	φ =	25.4	Shear Safety Factor	1.3
Span Length (m)	L =	1.58	Concrete Desity (kN/m3)	25

			Failure's angle	35	H2 =		225
Use ((2 φ 25) (Ast) =	981.71875	mm²	Ln/2 =	790	Length of Beam L		1.9
					Quantity	0.0779625	
A =	α.fc'.b -0.5*(α.fc')^2 .b/(Ec* εcu) + 0.4*√fc'.b	10559.61022	c =	129.657	β =	0.65	
B =	- 0.4*√fc'.b.H + As.Es.εcu	561718.4646	a =	84.2771	hx =	h * (1 + x / 2L)	
D =	- As.Es.εcu.d	-250348098.4		I =	(bh^3 / 12) * (1+ x / 2 L)^3		
	C = [α.fc'.b -0.5*(α.fc')^2 .b/(Ec* εcu)]* c^2 =				165.7977466	kN.m	
	T = 0.4*√fc'.b.H * c - 0.4*√fc'.b.*c^2 - As.Es.εcu * c + As.Es.εcu.d =				165.7977466	kN.m	

c =	129.657	r = (εcu-α.fc'/Ec)*(c / εcu) =	64.64333895
C1 =	α.fc' * b * r / 10^6	=	0.850867949
C2 =	0.5*α.fc' * (b * (c - r) -π 25^2) / 10^6	=	0.34172363
T1 =	0.4*√fc' * (b * (h-c) - π 25^2) / 10^6	=	0.025970378
T2 =	(As.Es.εcu.(d / c - 1))	=	1.243644061

dc=ds*F	
ds=180-(20+8+25/2) =	139.5
F=(1-3.04 tan)^(-0.608) <= 1.55 =	3.38383
Use F= 1.55	1.55
dc =	216.225

Calculation of Tensile strain(εf) & Stress (fpf) of Steel bar

εs = εcu *(d-c) / c =	0.006334	fs =	1266.802799	> fy =	420	Not OK
					Use fy =	420

Calculation of Flextural Strength (Mn) : -

Inclination angle of reinforcement = ζ	12.2551	
φ 8 (As2) = л ((8^2) / 4)) * 2	100.531	
Use fy for stirrups =	420	

Mn1 =	C1 * (c - r / 2)	82.8196			
Mn2 =	C2 * 2/3*(c - r)	14.8112	Vc =	(0.18*√fc'* (b*ds - (3.141592654*25^2)) / k11) / 1000	49.01986603
Mn3 =	T1 * (h - c)/2	0.65371	Vf =	((0.4*(√fc')*0.9* (b* ds - π 25^2) / k11) /(TAN(35*3.14159/180))) /1000	140.0154093
Mn4 =	As *fs *(d-c)	96.7484	Vs =	(0.9 *(As2 /800)* fy * ds /1.3) /1000)	0
		195.033	Dowel action Vda	0.2*As*fy*sinζ	17.50432583
					206.5396012

M D.L of Beam Self Weight	((b * H - 50 * 50)*G12 / 10^6 + (0.5 * L* 120 *b - ((L + 840)/2) *50*50)) / 10^9) *k12 * L^2 / 8) / 2)	0.14699		
	M total =	194.886		
M total = (Pu,f * shear span) / 2	Pu,f = (2 * M) /a	721.8	Vt =	413.0792023
	Theoritecal Failure's load =			413.0792023
	Experemental failure's load =			460
Pu,f > Vt — Shear Domiant	Accuracy of the Method % =			0.897998266

Nasser's formulas

CONTROL TAPERED BEAM (10)
2 φ 25 (W/O) Stirrups

			φ of stirrups (mm) =	8
Effective Depth (mm) for shear for tapered beam	(ds) =	216.225	θ	15.897
Total thickness	H (Total)	405	Tan (θ)	0.284801
Compressive Strength (Mpa)	fc' =	135	Effective Depth (at mid span) (mm) d	364.5
Beam Width (mm)	b =	150	Yield Stress of Steel main Reinf.(Mpa) fy	420
Beam's depth at support (mm)	H 1 =	180	Modulus of Elasticity of Steel (Mpa) Es	200000
Concrete Modulus of Elasticity (Mpa)	Ec =	50000	Ultimate Concrete Strain εcu	0.0035
Ratio of (α1) to (ɣ)	α =	0.65	Overhange length (m)	0.16
Ratio of Average concrete Stress to (fc')	α1 =	0.85	Load gap (m)	0.30
Concrete cover (mm)	c =	20	Shear Span (m)	0.640
Daimeter of main Reinforcemdnt (mm)	φ =	25	Shear Safety Factor	1.3
Span Length (m)	L =	1.58	Concrete Desity (kN/m3)	25

			Failure's angle	35	H2 =		225
Use ((2 φ 25) (Ast) =	981.71875	mm²	Ln/2 =	790	Length of Beam L		1.9
					Quantity	0.0779625	
A =	α.fc'.b -0.5*(α.fc')^2 .b/(Ec* εcu) + 0.4*√fc'.b	10559.61022	c =	129.6987703	β =	0.65	
B =	- 0.4*√fc'.b.H + As.Es.εcu	561718.4646	a =	84.30420067	hx =	h * (1 + x / 2L)	
D =	- As.Es.εcu.d	-250485539.1		I =	(bh^3 / 12) * (1+ x / 2 L)^3		
	C = [α.fc'.b -0.5*(α.fc')^2 .b/(Ec* εcu)]* c^2 =				165.904266	kN.m	
	T = 0.4*√fc'.b.H * c - 0.4*√fc'.b.*c^2 - As.Es.εcu * c + As.Es.εcu.d =				165.904266	kN.m	

c =	129.6987703	r = (εcu-α.fc'/Ec)*(c / εcu) =	64.66410117	ds=180-(20+8+25/2) =	139.5
C1 =	α.fc' * b * r / 10^6 =		0.851141232	F=(1-3.04 tan)^(-0.608) <= 1.55 =	3.383826
C2 =	0.5*α.fc' * (b * (c - r) -π 25^2) / 10^6 =		0.341861055	Use F= 1.55	1.55
T1 =	0.4*√fc' * (b * (h-c) - π 25^2) / 10^6 =		0.025941346	dc =	216.225
T2 =	(As.Es.εcu.(d / c - 1)) =		1.244083799		

dc=ds*F

Calculation of Tensile strain(εf) & Stress (fpf) of Steel bar

εs = εcu *(d-c) / c =	0.006336254	fs =	1267.250726	> fy =	420	Not OK
					Use fy =	420

Calculation of Flextural Strength (Mn) : -

			Inclination angle of reinforcement = ζ	12.28284898	
			φ 8 (As2) = л ((8^2) / 4)) * 2	100.53088	
Mn1 =	C1 * (c - r / 2)	82.8728297	Use fy for stirrups =	420	
Mn2 =	C2 * 2/3*(c - r)	14.82188038	Vc =	(0.18*√fc'* (b*ds - (3.141592654*25^2)) / k11) / 1000	49.01986603
Mn3 =	T1 * (h - c)/2	0.652440815	Vf =	((0.4*(√fc')*0.9* (b* ds - π 25^2) / k11) /(TAN(35*3.14159/180))) /1000	140.0154093
Mn4 =	As *fs *(d-c)	96.8136833	Vs =	(0.9 *(As2 /800)* fy * ds /1.3) /1000)	0
		195.1608342	Dowel action Vda	0.2*As*fy*sinζ	17.54328389
					206.5785592

M D.L of Beam Self Weight	((b * H - 50 * 50)*G12 / 10^6 + (0.5 * L* 120 *b - ((L + 840)/2) *50*50)) / 10^9) *k12 * L^2 / 8) / 2)		0.146993834		
		M total =	195.0138404		
M total = (Pu,f * shear span) / 2		Pu,f = (2 * M) /a	609.4182512	Vt =	413.1571184
		Theoritecal Failure's load =			413.1571184
		Experemental failure's load =			432
Pu,f > Vt	Shear Domiant	Accuracy of the Method % =			0.956382219

Concrete cover (mm)			*c =*	*20*	*TAPERED BEAM (11) (W/O) STIRRUPS*	*φ of stirrups(mm)*		*8*
Effective Depth for tapered beam's shear			*(ds) =*	*216.225*		*θ*		*15.897*
Total thickness			*H (Total)*	*405*		*Tan (θ)*		*0.2848009*
Compressive Strength (Mpa)			*fc' =*	*135*		*Effective Depth (at mid span) (mm) d*		*364.5*
Beam Width (mm)			*b =*	*150*		*Yield Stress of Steel Reinf.(Mpa) fy*		*420*
Beam's depth at support (mm)			*H 1 =*	*180*		*Modulus of Elasticity of Steel (Mpa) Es*		*200000*
Concrete Modulus of Elasticity (Mpa)			*Ec =*	*50000*		*Ultimate Concrete Strain εcu*		*0.0035*
Ratio of (α1) to (ɣ)			*α =*	*0.65*		*Overhange length (m)*		*0.16*
Ratio of Average concrete Stress to (fc')			*α1 =*	*0.85*		*Load gap (m)*		*0.30*
Daimeter of main Reinforcemdnt (mm)			*φ =*	*25*		*Shear Span (m)*		*0.640*
Span Length (m)			*L =*	*1.58*		*Shear Safety Factor*		*1.3*
						Concrete Desity (kN/m3)		*25*
Li	*Use ((2 φ 25) (Ast) =*		*981.71875*	*mm²*	*Failure's angle*	*35*	*H2 =*	*225*
283.5	*NS CFRP bar by De Lorenzis and Antonio Nanni*				*Ln/2 =*	*790*	*Length of Beam L*	*1.9*
289.5	*Height of shear-strengthened part of cross*			*340*	*Concret's cover C*	*20*	*Nominal shear strength provided by FRP*	
344.5	*Nominal rod diameter db = mm*			*6*	*Average bond strength (τb)*	*CFRP bars angle (α°) =*	*90*	
414.5	*Effective length of rod crossed by crack corresponding to tensile strain of 4000me (Li) = mm*			*292.5*	*Reduced length of FRP rods (dnet)*	*Modulus of elasticity of bar (Eb) = N/mm²*	*175000*	
333	*Spacing of NS CFRP bar (S) = mm*			*100*	*The sum of the effective lengths of all*			
						Quantity =	*0.0779625*	
A =	*α.fc'.b -0.5*(α.fc')^2 .b/(Ec* εcu) + 0.4*√fc'.b*				*10559.61022*	*c =*	*129.6987703*	
B =	*- 0.4*√fc'.b.H + As.Es.εcu*				*561718.4646*	*β =*	*0.65*	
D =	*- As.Es.εcu.d*				*-250485539.1*	*hx =*	*h * (1 + x / 2L)*	
					I =	*(bh^3 / 12) * (1+ x / 2 L)^3*	*dc=ds*F*	
	C = [α.fc'.b -0.5(α.fc')^2 .b/(Ec* εcu)]* c^2*				*165.904266*	*kN.m*	*ds=180-(20+8+25/2) =*	*139.5*
	T = 0.4√fc'.b.H*c-0.4*√fc'.b.*c^2-As.Es.εcu*c+As.Es.εcu.d*				*165.904266*	*kN.m*	*F=(1-3.04 tan)^(-0.608) <= 1.55*	*3.3838264*
							Use F= 1.55	*1.55*
c =			*129.6987703*	*r = (εcu-α.fc'/Ec)*(c / εcu)=*		*64.66410117*	*dc =*	*216.225*
C1 =	*α.fc' * b * r / 10^6 =*					*0.851141232*		
C2 =	*0.5*α.fc' * (b * (c - r) -π 25^2) / 10^6 =*					*0.341861055*		
T1 =	*0.4*√fc' * (b * (h-c) - π 25^2) / 10^6 =*					*0.025941346*		
T2 =	*(As.Es.εcu.(d / c - 1)) =*					*1.244083799*		

<u>Calculation of Tensile strain(εf) & Stress (fpf) of Steel bar</u>

*εs = εcu *(d-c) / c =*	*0.006336254*	*fs =*	*1267.2507*	*> fy =*	*420*	*Not OK*
					Use fy =	*420*
<u>Calculation of Flextural Strength (Mn) : -</u>				*Inclination angle of reinforcement = ζ*		*12.28284898*

					φ 8 (As2) = л ((8^2) / 4)) * 2		100.53088
Mn1 =	C1 * (c - r / 2)		82.8728297	Use fy for stirrups	420		
Mn2 =	C2 * 2/3*(c - r)		14.82188038	Vc	(0.18*√fc'*(b*ds-(π*25^2))/M11)/1000		49.02146742
Mn3 =	T1 * (h - c)/2		0.652440815	Vf	((0.4√fc'*0.9*(b*ds-π25²)/M11/tan(35*π/180)/1000		140.0154093
Mn4 =	As *fs *(d-c)		96.8136833	Vs	(0.9 *(As2 /800)* fy * ds /1.3) /1000)		0
			195.1608342	Dowel action Vda	1.2*As*fy*sinθ		17.54328389
							206.5801606
M D.LBeam Self Weight =		((b*H-50*50)*G12/10^6+(0.5*L*120*b-((L+840)/2)*50*50))/10^9)*M12*L^2/8)/2)					0.146993834
						M total =	195.0138404
				M total = (Pu,f * shear span) / 2		Pu,f = (2 * M) /a	609.4182512
Calculations of CFRP bars for shear capacity							
dnet = dr – 2× c					300		
Ltot min = dnet – S		if (dnet / 3) ≤ S ≤ dnet			200		
Ltot min = 2 * dnet – 4 * S		if (dnet / 4) ≤ S ≤ dnet / 3			200	Vn=Vc+Vs+Vda+Vsf +Vf	233.6324683
τb = 0.001 * (db *Eb) / Li					3.58974359		
V1F = 2 π * db * τb * Ltot m i n					27.05230769		
V2F = 2 p db tb Li		(dnet / 3) ≤ S ≤ (dnet / 2)		V2Fcontrols if Li>S. If Li<S V2F controls with the			39.564
				V2F = 2 π db τb (Li + dnet - 2 S)			53.09015385
If (dnet – 2S) < Li < S		V2F = 4 π db τb Li			if Li < dnet – 2S		79.128
(dnet / 4) < S < dnet/3		V1F controls if Li > dnet – 2s					
If Li < dnet – 2s, V2F controls with the value				V2F = 2 π db τb (Li + dnet - 2 S)		if S < Li < dnet – 2S	53.09015385
V2F = 2 π db τb (2 Li + dnet - 3 S)				if dnet – 3S < Li < S			79.128
V2F = 6 π db τb Li				if Li ≤ dnet – 3S			118.692
						Theoritecal Failure's load =	467.2649366
				Pu,f > Vt	Shear Domiant	Experemental failure's load =	590
						Accuracy	0.791974469

Concrete cover (mm)		c =	20	TAPERED BEAM (12) (W/O) STIRRUPS	φ of stirrups(mm)		8
Effective Depth for tapered beam's shear		(ds) =	216.225		θ		15.897
Total thickness		H (Total)	405		Tan (θ)		0.284800886
Compressive Strength (Mpa)		fc' =	135		Effective Depth (at mid span) (mm) d		364.5
Beam Width (mm)		b =	150		Yield Stress of Steel Reinf.(Mpa) fy		420
Beam's depth at support (mm)		H 1 =	180		Modulus of Elasticity of Steel (Mpa) Es		200000
Concrete Modulus of Elasticity (Mpa)		Ec =	50000		Ultimate Concrete Strain εcu		0.0035
Ratio of (α1) to (γ)		α =	0.65		Overhange length (m)		0.16
Ratio of Average concrete Stress to (fc')		α1 =	0.85		Load gap (m)		0.30
Daimeter of main Reinforcemdnt (mm)		φ =	25		Shear Span (m)		0.640
Span Length (m)		L =	1.58		Shear Safety Factor		1.3
					Concrete Desity (kN/m3)		25
Li	Use ((2 φ 25) (Ast) =	981.71875	mm²	Failure's angle	35	H2 =	225
283.5	NS CFRP bar by De Lorenzis and Antonio Nanni			Ln/2 =	790	Length of Beam L	1.9
289.5	Height of shear-strengthened part of cross section dr = H at ds = mm (not failure position) the high from bars edge		340	Concret's cover C	20	Nominal shear strength provided by FRP shear reinforcement (V1F , V2F)	
344.5	Nominal rod diameter db = mm		6	Average bond strength (τb)	CFRP bar angle (α°) =	45	
414.5	Effective length of rod crossed by crack corresponding to tensile strain of 4000me (Li) = mm		292.5	Reduced length of FRP rods (dnet)	Modulus of elasticity of bar (Eb) = N/mm²	175000	
333	Spacing of NS CFRP bar (S) = mm		100	The sum of the effective lengths of all the rods crossed by the crack. (Ltot min)			
					Quantity =	0.0779625	
A =	α.fc'.b -0.5*(α.fc')^2 .b/(Ec* εcu) + 0.4*√fc'.b			10559.61022	c =	129.6987703	
B =	- 0.4*√fc'.b.H + As.Es.εcu			561718.4646	β =	0.65	
D =	- As.Es.εcu.d			-250485539.1	hx =	h * (1 + x / 2L)	
				I =	(bh^3 / 12) * (1+ x / 2 L)^3	dc=ds*F	
	C = [α.fc'.b -0.5*(α.fc')^2 .b/(Ec* εcu)]* c^2			165.904266	kN.m	ds=180-(20+8+25/2) =	139.5
	T = 0.4*√fc'.b.H*c-0.4*√fc'.b.*c^2-As.Es.εcu*c+As.Es.εcu.d			165.904266	kN.m	F=(1-3.04 tan)^(-0.608) <= 1.55 =	3.383826421
						Use F= 1.55	1.55
c =		129.6987703	r = (εcu-α.fc'/Ec)*(c / εcu) =		64.66410117	dc =	216.225
C1 =	α.fc' * b * r / 10^6 =				0.851141232		

C2 =	***0.5*α.fc' * (b * (c - r) -π 25^2) / 10^6 =***	***0.341861055***
T1 =	***0.4*√fc' * (b * (h-c) - π 25^2) / 10^6 =***	***0.025941346***
T2 =	***(As.Es.εcu.(d / c - 1)) =***	***1.244083799***

<u>Calculation of Tensile strain(εf) & Stress (fpf) of Steel bar</u>

εs = εcu *(d-c) / c =	***0.006336254***	***fs =***	***1267.250726***	***> fy =***	***420***	***Not OK***
					Use fy =	***420***

<u>Calculation of Flextural Strength (Mn) : -</u>

Inclination angle of reinforcement = ζ	***12.28284898***
φ 8 (As2) = л ((8^2) / 4)) * 2	***100.53088***

Mn1 =	***C1 * (c - r / 2)***	***82.8728297***	***Use fy for stirrups***	***420***	
Mn2 =	***C2 * 2/3*(c - r)***	***14.82188038***	***Vc***	***(0.18*√fc'*(b*ds-(π*25^2))/M11)/1000***	***49.02146742***
Mn3 =	***T1 * (h - c)/2***	***0.652440815***	***Vf***	***((0.4√fc'*0.9*(b*ds-π25²)/M11/tan(35*π/180)/1000***	***140.0154093***
Mn4 =	***As *fs *(d-c)***	***96.8136833***	***Vs***	***(0.9 *(As2 /800)* fy * ds /1.3) /1000)***	***0***
		195.1608342	***Dowel action Vda***	***0.2*As*fy*sinθ***	***17.54328389***
					206.5801606

M D.LBeam Self Weight =	***((b*H-50*50)*G12/10^6+(0.5*L*120*b-((L+840)/2)*50*50))/10^9)*M12*L^2/8)/2)***		***0.146993834***
		M total =	***195.0138404***
	M total = (Pu,f * shear span) / 2	***Pu,f = (2 * M) /a***	***609.4182512***

Calculations of CFRP bars for shear capacity				
dnet = dr – 2× c		***300***		
Ltot min = dnet – S	***if (dnet / 3) ≤ S ≤ dnet***	***200***		
Ltot min = 2 * dnet – 4 * S	***if (dnet / 4) ≤ S ≤ dnet / 3***	***200***	***Vn=Vc+Vs+Vda+Vsf+Vf***	***233.6324683***
τb = 0.001 * (db *Eb) / Li		***3.58974359***		
V1F = 2 π * db * τb * Ltot m i n		***27.05230769***		

V2F = 2 p db tb Li	***(dnet / 3) ≤ S ≤ (dnet / 2)***	***V2Fcontrols if Li>S. If Li<S V2F controls with the value***	***39.564***
		V2F = 2 π db τb (Li + dnet - 2 S)	***53.09015385***
If (dnet – 2S) < Li < S	***V2F = 4 π db τb Li***	***if Li < dnet – 2S***	***79.128***
(dnet / 4) < S < dnet/3	***V1F controls if Li > dnet – 2s***		

If Li < dnet – 2s, V2F controls with the value	***V2F = 2 π db τb (Li + dnet - 2 S)***	***if S < Li < dnet – 2S***	***53.09015385***
V2F = 2 π db τb (2 Li + dnet - 3 S)	***if dnet – 3S < Li < S***		***79.128***
V2F = 6 π db τb Li	***if Li ≤ dnet – 3S***		***118.692***
		Theoritecal Failure's load =	***467.2649366***
Pu,f > Vt	***Shear Domiant***	***Experemental failure's load =***	***590***
		Accuracy	***0.791974469***

Concrete cover (mm)		c =	20	TAPERED BEAM (13) (W/O) Stirrups	φ of stirrups (mm)		8	
Effective Depth for tapered beam's shear		(ds) =	216.225		θ		15.897	
Total thickness		H (Total)	405		Tan (θ)		0.284800886	
Compressive Strength (Mpa)		fc' =	135		Effective Depth (at mid span)d		364.5	
Beam Width (mm)		b =	150		Yield Stress of Steel Reinf.(Mpa) fy		420	
Beam's depth at support (mm)		H 1 =	180		Modulus of Elasticity of Steel Es		200000	
Concrete Modulus of Elasticity (Mpa)		Ec =	50000		Ultimate Concrete Strain εcu		0.0035	
Ratio of (α1) to (γ)		α =	0.65		Overhange length (m)		0.16	
Ratio of Average concrete Stress to (fc')		α1 =	0.85		Load gap (m)		0.30	
Daimeter of main Reinforcemdnt (mm)		φ =	25		Shear Span (m)		0.640	
Span Length (m)		L =	1.58		Shear Safety Factor		1.3	
					Concrete Desity (kN/m3)		25	
Li	Use ((2 φ 25) (Ast) =		981.71875	Failure's angle	35	H2 =	225	
199.5	NS CFRP bar by De Lorenzis and Antonio Nanni			Ln/2 =	790	Length of Beam L	1.9	
244.5	Height of shear-strengthened part of cross section dr = H at ds = mm (not failure position) the high from bars edge		325	Concret's cover (C) =	20	Nominal shear strength provided by FRP shear reinforcement (V1F , V2F)		
284.5	Nominal rod diameter db = mm		6	Average bond strength (τb)		CFRP bar angle α °	30	
319.5	Effective length of rod crossed by crack corresponding to tensile strain of 4000me (Li) = mm		221.5	Reduced length of FRP rods (dnet)		Modulus of elasticity of bar (Eb) = N/mm²	175000	
262	Spacing of NS CFRP bar (S) = mm		100	The sum of the effective lengths of all the rods crossed by the crack. (Ltot min)				
						β =	0.65	
A =	α.fc'.b -0.5*(α.fc')^2 .b/(Ec* εcu) + 0.4*√fc'.b =				10559.61022	c =	129.6987703	
B =	- 0.4*√fc'.b.H + As.Es.εcu =				561718.4646	hx =	h * (1 + x / 2L)	
D =	- As.Es.εcu.d =				-250485539.1	I =	(bh^3 / 12) * (1+ x / 2 L)^3	
	C = [α.fc'.b -0.5*(α.fc')^2 .b/(Ec* εcu)]* c^2				165.904266	kN.m	Quantity =	0.0779625
	T = 0.4*√fc'.b.H * c - 0.4*√fc'.b.*c^2 - As.Es.εcu * c + As.Es.εcu.d				165.904266	kN.m		
c =		129.6987703	r = (εcu-α.fc'/Ec)*(c / εcu) =		64.66410117	dc=ds*F		
C1 =	α.fc' * b * r / 10^6 =				0.851141232	ds=180-(20+8+25/2) =	139.5	
C2 =	0.5*α.fc' * (b * (c - r) -π 25^2) / 10^6 =				0.341861055	F=(1-3.04 tan)^(-0.608) <=	3.383826421	
T1 =	0.4*√fc' * (b * (h-c) - π 25^2) / 10^6 =				0.025941346	Use F= 1.55	1.55	
T2 =	(As.Es.εcu.(d / c - 1)) =				1.244083799	dc =	216.225	

Calculation of Tensile strain(εf) & Stress (fpf) of Steel bar

εs = εcu *(d-c) / c =	0.006336254	fs =	1267.250726	> fy =	420	Not OK

Use fy =	420
Inclination angle of reinforcement = ζ	12.28284898
φ 8 (As2) = л ((8^2) / 4)) * 2	100.53088
Use fy for stirrups =	420

Calculation of Flextural Strength (Mn) : -

Mn1 =	C1 * (c - r / 2)	82.8728297			
Mn2 =	C2 * 2/3*(c - r)	14.82188038	Vc =	(0.18*√fc'* (b*ds - (л*25^2)) / I11) / 1000	49.01986603
Mn3 =	T1 * (h - c)/2	0.652440815	Vf =	((0.4*(√fc')*0.9* (b* ds - π 25^2) / I11) /(TAN(35*л/180))) /1000	140.0154093
Mn4 =	As *fs *(d-c)	96.8136833	Vs =	(0.9 *(As2 /800)* fy * ds /1.3) /1000)	0
		195.1608342	Dowel action Vda	0.2*As*fy*sinθ	17.54328389
					206.5785592

M D.L of Beam Self Weight =	((b * H - 50 * 50)*G12 / 10^6 + (0.5 * L* 120 *b - ((L + 840)/2) *50*50)) / 10^9) *I12 * L^2 / 8) / 2)	0.146993834

	M total =	195.0138404
M total = (Pu,f * shear span) / 2	Pu,f = (2 * M) /a	609.4182512

Calculations of CFRP bars for shear capacity				
dnet = dr – 2× c			285	
Ltot min = dnet – S		if (dnet / 3) ≤ S ≤ dnet	185	
Ltot min = 2 * dnet – 4 * S		if (dnet / 4) ≤ S ≤ dnet / 3	170	
τb = 0.001 * (db *Eb) / Li			4.740406321	
V1F = 2 π * db * τb * Ltot m i n			30.36514673	
V2F = 2 p db tb Li	(dnet / 3) ≤ S ≤ (dnet / 2)		V2F controls if Li > S. If Li < S V2F controls with	39.564
			V2F = 2 π db τb (Li + dnet - 2 S)	54.74657336
If (dnet – 2S) < Li < S	V2F = 4 π db τb Li		if Li < dnet – 2S	79.128
(dnet / 4) < S < dnet/3	V1F controls if Li > dnet – 2s			
If Li < dnet – 2s, V2F controls with the value		V2F = 2 π db τb (Li + dnet - 2 S)	if S < Li < dnet – 2S	54.74657336
V2F = 2 π db τb (2 Li + dnet - 3 S)		if dnet – 3S < Li < S		76.44872235
V2F = 6 π db τb Li		if Li ≤ dnet – 3S		118.692

		Vn = Vc + Vs +Vda +Vsf+ Vf =	236.9437059
		Theoritecal Failure's load =	473.8874119
		Experemental failure's load =	585
Pu,f > Vt	Shear Domiant	Accuracy of the Method =	0.810063952

			TAPERED BEAM (14) (W/O) Stirrups	φ of stirrups (mm)	8
Effective Depth of tapered beam's shear	(ds) =	216.225		θ	15.897
Total thickness	H (Total)	405		Tan (θ)	0.284800886
Compressive Strength (Mpa)	fc' =	135		Effective Depth (at mid span) (mm) d	364.5
Beam Width (mm)	b =	150		Yield Stress of Steel -main Reinf.(Mpa) fy	420
Beam's depth at support (mm)	H 1 =	180		Modulus of Elasticity of Steel (Mpa) Es	200000
Concrete Modulus of Elasticity (Mpa)	Ec =	50000		Ultimate Concrete Strain εcu	0.0035
Ratio of (α1) to (ɣ)	α =	0.65		Overhange length (m)	0.16
Ratio of Average concrete Stress to (fc')	α1 =	0.85		Load gap (m)	0.30
Concrete cover (mm)	c =	20		Shear Span (m)	0.640
Daimeter of main Reinforcemdnt (mm)	φ =	25		Shear Safety Factor	1.3
Span Length (m)	L =	1.58		Concrete Desity (kN/m3)	25
		Failure's angle	35	H2 =	225
Use ((2 φ 25) (Ast) =	981.71875	mm²	Ln/2 =	790 — Length of Beam L =	1.9

CFRP Strips	dfv =	169.5	Ultimate tensile strength (ffu*) (N/mm2) =	3600	number of plies of FRP (n) =	1
	Width of each sheet wf =	50	Rupture strain εfu* =	0.015	CFRP strip Orientation angle (α °) =	90
	Span between each sheet sf =	100	Modulus of elasticity (Ef) (N/mm2) =	175000	Shear reduction factor (φ) =	0.75
	Thickness per ply tf =	0.165	Environment reduction factor CE =	0.95	single FRP lenth on one face =	210
			FRP strength reduction factor (ψf) =	0.85	Quantity =	0.0779625

A =	α.fc'.b -0.5*(α.fc')^2 .b/(Ec* εcu) + 0.4*√fc'.b	10559.61022	c =	129.6987703	β =	0.65
B =	- 0.4*√fc'.b.H + As.Es.εcu	561718.4646		hx =	h * (1 + x / 2L)	
D =	- As.Es.εcu.d	-250485539.1				
				I =	(bh^3 / 12) * (1+ x / 2 L)^3	

C = [α.fc'.b -0.5*(α.fc')^2 .b/(Ec* εcu)]* c^2	165.904266	kN.m	dc=ds*F	
T = 0.4*√fc'.b.H * c - 0.4*√fc'.b.*c^2 - As.Es.εcu * c + As.Es.εcu.d	165.904266	kN.m	ds=180-(20+8+25/2) =	139.5
			F=(1-3.04 tan)^(-0.608)	<=1.55 3.358826421
c = 129.6987703 — r = (εcu-α.fc'/Ec)*(c / εcu) =	64.66410117		Use F= 1.55	1.55
C1 = α.fc' * b * r / 10^6 =	0.851141232		dc =	216.225
C2 = 0.5*α.fc' * (b * (c - r) -π 25^2) / 10^6 =	0.341861055			
T1 = 0.4*√fc' * (b * (h-c) - π 25^2) / 10^6 =	0.025941346			
T2 = (As.Es.εcu.(d / c - 1)) =	1.244083799			

<u>*Calculation of Tensile strain(εf) & Stress (fpf) of Steel bar*</u>

εs = εcu *(d-c) / c =	0.006336254	fs =	1267.250726	> fy =	420	Not OK

Use fy =	*420*
Inclination angle of reinforcement= ζ	*12.28284898*
*φ 8 (As2) = л ((8^2) / 4)) * 2*	*100.53088*
Use fy for stirrups =	*420*

Calculation of Flextural Strength (Mn) : -

Mn1	*C1 * (c - r / 2)*	*82.8728297*			
Mn2	*C2 * 2/3*(c - r)*	*14.82188038*	*Vc =*	*(0.18*√fc'* (b*ds - (π*25^2)) / J12) / 1000*	*49.01986603*
Mn3	*T1 * (h - c)/2*	*0.652440815*	*Vf =*	*((0.4*(√fc')*0.9* (b* ds - π 25^2) / J12) /(TAN(35*π/180))) /1000*	*140.0154093*
Mn4	*As *fs *(d-c)*	*96.8136833*	*Vs =*	*(0.9 *(As2 /800)* fy * ds /1.3) /1000)*	*0*
		195.1608342	*Dowel action Vda=*	*0.2*As*fy*sinθ*	*17.54328389*
					206.5785592
M D.L of Beam Self Weight	*((b * H - 50 * 50)*G12 / 10^6 + (0.5 * L* 120 *b - ((L + 840)/2) *50*50)) / 10^9) *J13 * L^2 / 8) / 2)*				*0.146993834*
				M total =	*195.0138404*
			*M total = (Pu,f * shear span) / 2*	*Pu,f = (2 * M) /a*	*609.4182512*

Calculations of CFRP strips for shear capacity						
ffu = CE ffu =*	*3.42*		*εfu = CEεfu* =*	*0.01425*	*φVn = φ(Vc + Vs +Vda +Vsf+ ψf Vf) =*	*167.4243509*
Le = 23300 / (n tf Ef)^(0.58) =	*60.29109411*		*k1 = (fc' / 27)^(2/3) =*	*2.924017738*	*Pus =*	*334.8487018*
k2 = (dfv - Le) / dfv =	*0.64430033*		*κv = (k1k2Le / (11900 * εfu)) < 0.75 =*	*0.669822445*	*Experemental failure's load =*	*441*
εfe = κv εfu ≤ 0.004	*0.00954497*	*0.004*	*Afv = 2n tf wf =*	*16.5*	*Accuracy of the Method =*	*0.759294108*
ffe = εfeEf =	*0.7*		*Vf = (Afv ffe (sinα α + cos α) dfv) / sf =*	*19.59283368*		
					Pu,f > Vt	*Shear Domiant*

			TAPERED BEAM (15) (W/O)Stirrups	*φ of stirrups (mm)*	*8*
Effective Depth of tapered beam's shear	*(ds) =*	*216.225*		*θ*	*15.897*
Total thickness	*H (Total)*	*405*		*Tan (θ)*	*0.284800886*
Compressive Strength (Mpa)	*fc' =*	*135*		*Effective Depth (at mid span) (mm) d*	*364.5*
Beam Width (mm)	*b =*	*150*		*Yield Stress of Steel Reinf.(Mpa) fy*	*420*
Beam's depth at support (mm)	*H 1 =*	*180*		*Modulus of Elasticity of Steel (Mpa) Es*	*200000*
Concrete Modulus of Elasticity (Mpa)	*Ec =*	*50000*		*Ultimate Concrete Strain εcu*	*0.0035*
Ratio of (α1) to (ɣ)	*α =*	*0.65*		*Overhange length (m)*	*0.16*
Ratio of Average concrete Stress to (fc')	*α1 =*	*0.85*		*Load gap (m)*	*0.30*
Concrete cover (mm)	*c =*	*20*		*Shear Span (m)*	*0.640*
Daimeter of main Reinforcemdnt (mm)	*φ =*	*25*		*Shear Safety Factor*	*1.3*
Span Length (m)	*L =*	*1.58*		*Concrete Desity (kN/m3)*	*25*
		Failure's angle	*35*	*H2 =*	*225*

Use ((2 φ 25) (Ast) =			*981.71875*	*mm²*	*Ln/2 =*	*790*	*Length of Beam L =*	*1.9*
CFRP Strips	*dfv =*	*439.5*	*Ultimate tensile strength (ffu*) (N/mm2)*		*3600*	*Number of plies of FRP (n) =*		*1*
	Width of each sheet wf =	*50*	*Rupture strain εfu**		*0.015*	*CFRP strip Orientation angle (α °)*		*45*
	Span between each sheet sf =	*100*	*Modulus of elasticity (Ef) (N/mm2)*		*175000*	*Shear reduction factor (φ) =*		*0.75*
	Thickness per ply tf =	*0.165*	*Environment reduction factor CE*		*0.95*	*single FRP lenth on one face =*		*480*
			FRP strength reduction factor (ψf)		*0.85*	*Quantity =*		*0.0779625*
A =	*α.fc'.b -0.5*(α.fc')^2 .b/(Ec* εcu) + 0.4*√fc'.b =*				*10559.61022*	*c =*	*129.6987703*	
B =	*- 0.4*√fc'.b.H + As.Es.εcu =*				*561718.4646*	*β =*	*0.65*	
D =	*- As.Es.εcu.d =*				*-250485539.1*	*hx =*	*h * (1 + x / 2L)*	
					I =	*(bh^3 / 12) * (1+ x / 2 L)^3*		
	C = [α.fc'.b -0.5(α.fc')^2 .b/(Ec* εcu)]* c^2*			*165.904266*	*kN.m*		*dc=ds*F*	
	T = 0.4√fc'.b.H * c - 0.4*√fc'.b.*c^2 - As.Es.εcu * c + As.Es.εcu.d*			*165.904266*	*kN.m*		*ds=180-(20+8+25/2) =*	*139.5*
							F=(1-3.04 tan)^(-0.608)	[illegible]
c =		*129.6987703*	*r = (εcu-α.fc'/Ec)*(c / εcu) =*		*64.66410117*		*Use F= 1.55*	*1.55*
C1 =	*α.fc' * b * r / 10^6 =*				*0.851141232*		*dc =*	*216.225*
C2 =	*0.5*α.fc' * (b * (c - r) -π 25^2) / 10^6 =*				*0.341861055*			
T1 =	*0.4*√fc' * (b * (h-c) - π 25^2) / 10^6 =*				*0.025941346*			
T2 =	*(As.Es.εcu.(d / c - 1)) =*				*1.244083799*			

Calculation of Tensile strain(εf) & Stress (fpf) of Steel bar

*εs = εcu *(d-c) / c =*	*0.006336254*	*fs =*	*1267.250726*	*> fy =*	*420*	*Not OK*
				Use fy =	*420*	

Calculation of Flextural Strength (Mn) : -

Inclination angle of reinforcement = ζ	***12.28284898***	
φ 8 (As2) = л ((8^2) / 4)) * 2	***100.53088***	
Use fy for stirrups =	***420***	

Mn1	***C1 * (c - r / 2)***	***82.8728297***				
Mn2	***C2 * 2/3*(c - r)***	***14.82188038***	***Vc =***	***(0.18*√fc'* (b*ds - (3.141592654*25^2)) / I12) / 1000***	***49.01986603***	
Mn3	***T1 * (h - c)/2***	***0.652440815***	***Vf =***	***((0.4*(√fc')*0.9* (b* ds - π 25^2) / I12) /(TAN(35*3.14159/180))) /1000***	***140.0154093***	
Mn4	***As *fs *(d-c)***	***96.8136833***	***Vs =***	***(0.9 *(As2 /800)* fy * ds /1.3) /1000)***	***0***	
		195.1608342	***Dowel action Vda =***	***0.2*As*fy*sinθ***	***17.54328389***	
					206.5785592	

M Dead Load of Beam Self Weight =	***((b * H - 50 * 50)*G12 / 10^6 + (0.5 * L* 120 *b - ((L + 840)/2) *50*50)) / 10^9) *I13 * L^2 / 8) / 2)***	***0.146993834***

	M total =	***195.0138404***
M total = (Pu,f * shear span) / 2	***Pu,f = (2 * M) /a***	***609.4182512***

Calculations of CFRP strips for shear capacity

ffu = CE ffu* =	***3.42***		***εfu = CEεfu* =***	***0.01425***	***φVn = φ (Vc + Vs +Vda +Vsf+ ψf Vf)***		***200.6991881***
Le = 23300 / (n tf Ef)^(0.58) =	***60.29109411***		***k1 = (fc' / 27)^(2/3) =***	***2.924017738***		***Pus =***	***401.3983761***
k2 = (dfv - Le) / dfv =	***0.862818898***		***κv = (k1k2Le / (11900 * εfu)) < 0.75 =***	***0.896997001***	***0.75***	***Experemental failure's load***	***487***
εfe = κv εfu ≤ 0.004	***0.0106875***	***0.004***	***Afv = 2n tf wf =***	***16.5***		***Method's Accuracy***	***0.824226645***
ffe = εfeEf =	***0.7***		***Vf = (Afv ffe (sinα α + cos α) dfv) / sf =***	***71.78865672***			
					Pu,f > Vt	***Shear Domiant***	

			TAPERED BEAM (16) (W/O) Stirrups	φ of stirrups (mm)	8
Effective Depth of tapered beam's shear	(ds) =	216.225		θ	15.897
Total thickness	H (Total)	405		Tan (θ)	0.284800886
Compressive Strength (Mpa)	fc' =	135		Effective Depth (at mid span) (mm) d	364.5
Beam Width (mm)	b =	150		Yield Stress of Steel Reinf.(Mpa) fy	420
Beam's depth at support (mm)	H 1 =	180		Modulus of Elasticity of Steel (Mpa) Es	200000
Concrete Modulus of Elasticity (Mpa)	Ec =	50000		Ultimate Concrete Strain εcu	0.0035
Ratio of (α1) to (γ)	α =	0.65		Overhange length (m)	0.16
Ratio of Average concrete Stress to (fc')	α1 =	0.85		Load gap (m)	0.30
Concrete cover (mm)	c =	20		Shear Span (m)	0.640
Daimeter of main Reinforcemdnt (mm)	φ =	25		Shear Safety Factor	1.3
Span Length (m)	L =	1.58		Concrete Desity (kN/m3)	25
		Failure's angle	35	H2 =	225

Use ((2 φ 25) (Ast) =	981.71875	Ln/2 =	790	Length of Beam L	1.9

CFRP Strips	dfv =	309.5	Ultimate tensile strength (ffu*) (N/mm2)	3500	Number of plies of FRP reinforcement (n) =	1
	Width of each sheet wf =	50	Rupture strain εfu*	0.015	CFRP strip Orientation angle (α °) =	30
	Span between each sheet sf =	100	Modulus of elasticity (Ef) (N/mm2)	175000	Shear reduction factor (φ) =	0.75
	Thickness per ply tf =	0.165	Environment reduction factor CE	0.95	single FRP lenth on one face =	350
			FRP strength reduction factor (ψf)	0.85	Quantity =	0.0779625

A =	α.fc'.b -0.5*(α.fc')^2 .b/(Ec* εcu) + 0.4*√fc'.b =	10559.61022	c =	129.6987703
B =	- 0.4*√fc'.b.H + As.Es.εcu =	561718.4646	β =	0.65
D =	- As.Es.εcu.d =	-250485539.1	hx =	h * (1 + x / 2L)
		I =	(bh^3 / 12) * (1+ x / 2 L)^3	

C = [α.fc'.b -0.5*(α.fc')^2 .b/(Ec* εcu)]* c^2 =	165.904266	kN.m
T = 0.4*√fc'.b.H * c - 0.4*√fc'.b.*c^2 - As.Es.εcu * c + As.Es.εcu.d =	165.904266	kN.m

c =	129.6987703	r = (εcu-α.fc'/Ec)*(c / εcu) =	64.66410117
C1 =	α.fc' * b * r / 10^6 =		0.851141232
C2 =	0.5*α.fc' * (b * (c - r) -π 25^2) / 10^6 =		0.341861055
T1 =	0.4*√fc' * (b * (h-c) - π 25^2) / 10^6 =		0.025941346
T2 =	(As.Es.εcu.(d / c - 1)) =		1.244083799

dc=ds*F	
ds=180-(20+8+25/2) =	139.5
F=(1-3.04 tan)^(-0.608) <= 1.55	1.333826421
Use F= 1.55	1.55
dc =	216.225

Calculation of Tensile strain(εf) & Stress (fpf) of Steel bar

εs = εcu *(d-c) / c =	0.006336254	fs =	1267.250726	> fy =	420	Not OK
				Use fy =	420	
			Inclination angle of reinforcement = ζ		12.28284898	

Calculation of Flextural Strength (Mn) : -

				φ 8 (As2) = л ((8^2) / 4)) * 2	100.53088
Mn1 =	C1 * (c - r / 2)	82.8728297		Use fy for stirrups =	420
Mn2 =	C2 * 2/3*(c - r)	14.82188038	Vc =	(0.18*√fc'* (b*ds - (л*25^2)) / I12) / 1000	49.01986603
Mn3 =	T1 * (h - c)/2	0.652440815	Vf =	((0.4*(√fc')*0.9* (b* ds - π 25^2) / I12) /(TAN(35*л/180))) /1000	140.0154093
Mn4 =	As *fs *(d-c)	96.8136833	Vs =	(0.9 *(As2 /800)* fy * ds /1.3) /1000)	0
		195.1608342	Dowel action Vda =	0.2*As*fy*sinθ	17.54328389
					206.5785592
M Dead Load of Beam Self Weight =	((b * H - 50 * 50)*G12 / 10^6 + (0.5 * L* 120 *b - ((L + 840)/2) *50*50)) / 10^9) *I13 * L^2 / 8) / 2)				0.146993834
				M total =	195.0138404
			M total = (Pu,f * shear span) / 2	Pu,f = (2 * M) /a	609.4182512

Calculations of CFRP strips for shear capacity						
ffu = CE * ffu =	3.325		εfu = Ce * εfu =	0.01425	φVn = φ (Vc + Vs +Vda +Vsf+ ψf Vf) =	186.0618821
Le = 23300 / (n tf Ef)^(0.58) =	60.29109411		k1 = (fc' / 27)^(2/3) =	2.924017738	Pus =	372.1237642
k2 = (dfv - Le) / dfv =	0.805198404		κv = (k1k2Le / (11900 * εfu)) < 0.75 =	0.837094035	Experemental failure's load =	465
εfe = κv εfu ≤ 0.004	0.01192859	0.004	Afv = 2n tf wf =	16.5	Accuracy of the Method =	0.80026616
ffe = εfeEf =	0.7		Vf = (Afv ffe (sinα α + cos α) dfv) / sf =	48.82817674		
					Pu,f > Vt	Shear Domiant

				TAPERED BEAMS (17 + 18) WITH STIRRUPS	φ of stirrups (mm)		8
Effective Depth for shear for tapered beam at failure's position		(ds) =	216.225		θ		15.897
Total thickness		H (Total)	405		Tan (θ)		0.284800886
Compressive Strength (Mpa)		fc' =	135		Effective Depth (at mid span) (mm) d		364.5
Beam Width (mm)		b =	150		Yield Stress of Steel Reinf.(Mpa) fy		420
Beam's depth at support (mm)		H 1 =	180		Modulus of Elasticity of Steel (Mpa) Es		200000
Concrete Modulus of Elasticity		Ec =	50000		Ultimate Concrete Strain εcu		0.0035
Ratio of (α1) to (γ)		α =	0.65		Overhange length (m)		0.16
Ratio of Average concrete Stress (fc')		α1 =	0.85		Load gap (m)		0.30
Concrete cover (mm)		c =	20		Shear Span (m)		0.640
Daimeter of main Reinforcemdnt (mm)		φ =	25		Shear Safety Factor		1.3
Span Length (m)		L =	1.58		Concrete Desity (kN/m3)		25
			Failure's angle 4	35	H2 =		225
Use ((2 φ 25) (Ast) =		981.71875	Failure's angle 5	35	Whole Length of Beam L	1.9	
					Ln/2 =	790	
A =	α.fc'.b -0.5*(α.fc')^2 .b/(Ec* εcu) + 0.4*√fc'.b =				10559.61022	c =	129.6987703
B =	- 0.4*√fc'.b.H + As.Es.εcu =				561718.4646	β =	0.65
D =	- As.Es.εcu.d =				-250485539.1		
	C = [α.fc'.b -0.5*(α.fc')^2 .b/(Ec* εcu)]* c^2			165.904266	kN.m		
	T = 0.4*√fc'.b.H * c - 0.4*√fc'.b.*c^2 - As.Es.εcu * c + As.Es.εcu.d			165.904266	kN.m		
c =		129.6987703	r = (εcu-α.fc'/Ec)*(c / εcu) =		64.66410117		
C1 =	α.fc' * b * r / 10^6 =				0.851141232		
C2 =	0.5*α.fc' * (b * (c - r) -π 25^2) / 10^6 =				0.341861055		
T1 =	0.4*√fc' * (b * (h-c) - π 25^2) / 10^6 =				0.025941346		
T2 =	(As.Es.εcu.(d / c - 1)) =				1.244083799		

Calculation of Tensile strain(εf) & Stress (fpf) of Steel bar

	εs = εcu *(d-c) / c =	0.006336254	fs =	1267.250726	> fy =	420	Not OK
						Use fy =	420

Calculation of Flextural Strength (Mn) : -

					Inclination angle of reinforcement = ζ		12.28284898
					φ 8 (As2) = л ((8^2) / 4)) * 2		100.53088
Mn1 =	C1 * (c - r / 2)	82.8728297			Use fy for stirrups =		400
Mn2 =	C2 * 2/3*(c - r)	14.82188038	Vc =	(0.18*√fc'* (b*ds - (л*25^2)) / I11) / 1000		49.01986603	
Mn3 =	T1 * (h - c)/2	0.652440815	Vf =	((0.4*(√fc')*0.9*(b*ds-π 25^2)/I11)/(tan(35*л/180)))/1000		140.0154093	
Mn4 =	As *fs *(d-c)	96.8136833	Vs =	(0.9 *(As2 /100)* fy * ds /1.3) /1000) 5 stirrups.		60.195571	
		195.1608342	Dowel action Vda	0.2*As*fy*sinθ		17.54328389	
					∑	266.7741302	

M D.L of Beam Self Weight =	***((b*H-50*50)*G12/10^6+(0.5*L*120*b-((L+840)/2)*50*50))/10^9)*I12*L^2/8)/2)***	***0.146993834***
	M total =	***195.0138404***
M total = (Pu,f * shear span) / 2	***Pu,f = (2 * M) /a***	***609.4182512***
	Vt =	***533.5482604***
	Theoritecal Failure's load	***533.5482604***
	Experemental failure's load	***565***
	Accuracy of the Method =	***0.944333204***
(0.9 *(As2 /100)* fy * ds /1.3) /1000) 4 stirrups		***42.99683643***
	∑	***249.5753956***
	Vt =	***499.1507913***
Theoritecal Failure's load =		***499.1507913***
Experemental failure's load =		***518***
Accuracy of the Method =		***0.963611566***
Pu,f > Vt	***Shear Domiant***	
	dc=ds*F	
	ds=180-(20+8+25/2) =	***139.5***
	F=(1-3.04 tan)^(-0.608) <= 1.55 =	***3.383826421***
	Use F= 1.55	***1.55***
	dc =	***216.225***

Nasser's formulas

			TAPERED BEAMS (19) (W/O) Stirrups	φ of stirrups (mm) =	8
Effective Depth (mm) for shear for tapered beam	(ds) =	187.55		θ	15.897
Total thickness	H (Total)	405		Tan (θ)	0.2848
Compressive Strength (Mpa)	fc' =	135		Effective Depth (at mid span) (mm) d	346
Beam Width (mm)	b =	150		Yield Stress of main Reinf.(Mpa) fy	420
beam depth at support (mm)	H 1 =	180		Modulus of Elasticity of Steel Es	200000
Concrete Modulus of Elasticity (Mpa)	Ec =	50000		Ultimate Concrete Strain εcu	0.0035
Ratio of (α1) to (γ)	α =	0.65		Overhang length (m)	0.16
Ratio of Average concrete Stress to (fc')	α1 =	0.85		Load gap (m)	0.30
Concrete cover (mm)	c =	20		Shear Span (m)	0.640
Daimeter of main Reinforcemdnt (mm)	φ =	16		Shear Safety Factor	1.3
Span Length (m)	L =	1.58		Concrete Desity (kN/m3)	25
		Failure's angle	35	H2 = 225	Quantity 0.07796

Use ((4 φ 16) (Ast) =	804.224	mm²	Ln/2 =	790	Length of Beam L =	1.9
A =	α.fc'.b -0.5*(α.fc')^2 .b/(Ec* εcu) =	9862.473214	c =	114.86283	β =	0.65
B =	As.Es.εcu =	562956.8	a =	74.66083948		
D =	- As.Es.εcu.d =	-194783052.8	hx =	h * (1 + x / 2L)		
	C = [α.fc'.b -0.5*(α.fc')^2 .b/(Ec* εcu)]* c^2 =			130.1202416	kN.m	
	T = - As.Es.εcu * c + As.Es.εcu.d =			130.1202416	kN.m	

c = 114.863	r = (εcu-α.fc'/Ec)*(c / εcu) =	57.26732523	dc=ds*F	
C1 =	α.fc' * b * r / 10^6 =	0.753781168	ds=180-(20+8+16+15) =	121
C2 =	0.5*α.fc' * (b * (c - r) -π 25^2) / 10^6 =	0.292902055	F=(1-3.04 tan)^(-0.608) <= 1.55 =	3.38383
T1 =		0	Use F= 1.55	1.55
T2 =	(As.Es.εcu.(d / c - 1)) =	1.132831584	dc =	187.55

Calculation of Tensile strain(εf) & Stress (fpf) of Steel bar

εs = εcu *(d-c) / c =	0.00704301	fs =	1408.602061	> fy =	420	Not OK
				Use fy =	420	

Calculation of Flextural Strength (Mn) : -

			Inclination angle of reinforcement = ζ	11.86673527	
			φ 8 (As2) = л ((8^2) / 4)) * 2	100.53088	
Mn1	C1 * (c - r / 2)	64.9979	Use fy for stirrups =	420	
Mn2	C2 * 2/3*(c - r)	11.2466	Vc =	(0.18*√fc'* (b*ds - (3.14*25^2)) / M12) / 1000	42.1017
			Vf =		0
Mn4	As *fs *(d-c)	78.0721	Vs =		0
		154.317	Dowel action Vda =	0.2*As*fy*sinζ	13.8848
					55.9865

M D.L of Beam Self Weight	((b * H - 50 * 50)*G12 / 10^6 + (0.5 * L* 120 *b - ((L + 840)/2) *50*50)) / 10^9) *M13 * L^2 / 8) / 2)	0.146993834

		M total =	*154.1696348*		
	*M total = (Pu,f * shear span) / 2*	*Pu,f = (2 * M) /a*	*481.7801086*	*Vt =*	*111.973*
		Theoritecal Failure's load =			*111.973*
		Experemental failure's load =			*111*
Pu,f > Vt	*Shear Domiant*	*Accuracy of the Method =*			*1.00877*

Nasser's formulas			TAPERED BEAM (20) (W/O) Stirrups	φ of stirrups (mm) =	8
Effective Depth (mm) for shear for tapered beam	(ds) =	187.55		θ	15.897
Total thickness	H (Total)	405		Tan (θ)	0.2848
Compressive Strength (Mpa)	fc' =	135		Effective Depth (at mid span) (mm) d	336
Beam Width (mm)	b =	150		Yield Stress of main Reinf.(Mpa) fy	420
beam depth at support (mm)	H 1 =	180		Modulus of Elasticity of Steel Es	200000
Concrete Modulus of Elasticity (Mpa)	Ec =	50000		Ultimate Concrete Strain εcu	0.0035
Ratio of (α1) to (ɣ)	α =	0.65		Overhange length (m)	0.16
Ratio of Average concrete Stress to (fc')	α1 =	0.85		Load gap (m)	0.30
Concrete cover (mm)	c =	20		Shear Span (m)	0.640
Daimeter of main Reinforcemdnt (mm)	φ =	16		Shear Safety Factor	1.3
Span Length (m)	L =	1.58		Concrete Desity (kN/m3)	25
		Failure's angle	35	H2 = 225	Quantity 0.07796

Use ((4 φ 16) (Ast) =	804.224	mm²	Ln/2 =	790	Length of Beam L =	1.9
A =	α.fc'.b -0.5*(α.fc')^2 .b/(Ec* εcu) + 0.4*√fc'.b =	10559.61022	c =	114.7182683	β =	0.65
B =	- 0.4*√fc'.b.H + As.Es.εcu =	437472.1396	a =	74.56687438		
D =	- As.Es.εcu.d =	-189153484.8	hx =	h * (1 + x / 2L)		
	C = [α.fc'.b -0.5*(α.fc')^2 .b/(Ec* εcu)]* c^2 =			129.7929196	kN.m	
	T = 0.4*√fc'.b.H * c - 0.4*√fc'.b.*c^2 - As.Es.εcu * c + As.Es.εcu.d =			129.7929196	kN.m	

c = 114.718268	r = (εcu-α.fc'/Ec)*(c / εcu) =	57.1952509	dc=ds*F	
C1 =	α.fc' * b * r / 10^6 =	0.75283249	ds=180-(20+8+16+15) =	121
C2 =	0.5*α.fc' * (b * (c - r) -π 25^2) / 10^6 =	0.292424997	F=(1-3.04 tan)^(-0.608) <= 1.55 =	3.38383
T1 =	0.4*√fc' * (b * (h-c) - π 25^2) / 10^6 =	0.036384809	Use F= 1.55	1.55
T2 =	(As.Es.εcu.(d / c - 1)) =	1.085895537	dc =	187.55

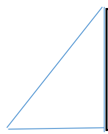
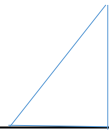
Calculation of Tensile strain(εf) & Stress (fpf) of Steel bar

εs = εcu *(d-c) / c =	0.006751201	fs =	1350.240154	> fy =	420	Not OK
				Use fy =	420	

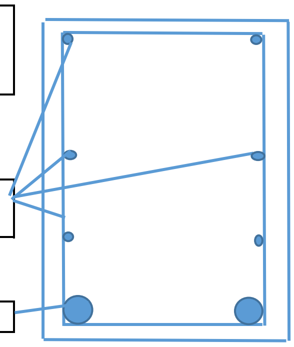
Calculation of Flextural Strength (Mn) : -

			Inclination angle of reinforcement = ζ	11.86673527	
			φ 8 (As2) = л ((8^2) / 4)) * 2	100.53088	
Mn1	C1 * (c - r / 2)	64.834418	Use fy for stirrups =	420	
Mn2	C2 * 2/3*(c - r)	11.2141121	Vc =	(0.18*√fc'* (b*ds - (3.141592654*25^2)) / M12) / 1000	42.10011096
Mn3	T1 * (h - c)/2	1.18763166	Vf =	((0.4*(√fc')*0.9* (b* ds - π 25^2) / M12) /(TAN(35*3.14159/180))) /1000	120.2505188
Mn4	As *fs *(d-c)	74.7432334	Vs =	(0.9 *(As2 /800)* fy * ds /1.3) /1000)	0
		151.979395	Dowel action Vda =	0.2*As*fy*sinζ	13.89169387
					176.2423236

M D.L of Beam Self Weight	((b * H - 50 * 50)*G12 / 10^6 + (0.5 * L* 120 *b - ((L + 840)/2) *50*50)) / 10^9) *M13 * L^2 / 8) / 2)	0.146993834		
	M total =	151.8324013		
M total = (Pu,f * shear span) / 2	Pu,f = (2 * M) /a	474.476254	Vt =	352.485
	Theoritecal Failure's load =			352.485
	Experemental failure's load =			281
Pu,f > Vt	Shear Domiant	Accuracy of the Method =		1.25439

Deep Beam Method					UHPCTB 1				
H t =			405	mm					
H1 =			180	mm					
d1 =			216.225	mm					
d2 =			337		Cover	20	mm		
H 2 =			225	mm	bar =	25	mm		
Load gap =			0.2	m	bar 2 =	8	mm		
a =			0.69	mm					
b =			150	mm	dc=ds*F				
L t =			1.9	m	ds=180-(20+8+25/2) =				139.5
Overhange length (m) =			0.16	m					
Ln =			1.58	m	F=(1-3.04 tan)^(-0.608) <= 1.55				3.38383
fc' =			135	Mpa	Use F= 1.55				1.55
fy =			420	Mpa	dc =				216.225
Applied load =			215	kN					
P L = Applied load =			215	kN				Two point loads	
First : - Calculate the factored load: -					Theoritecal FLAXURAL Failure's load=			346.432	kN
P D =			2.027025	kN	Maximum Capasity of testing device =			600	kN
Pu =			346.43243	kN	Experemental SHEAR failure's load =			416	kN
R A = R B =			173.216215	kN	Highest shear failure load			490	
Second: - Check if the beam is deep								1.41442	
Clear span , Ln =		1.58	(Ln/H)*1000	3.90123	< 4 OK Deep beam				
Shear span , a =		0.69	(a/H)*1000	1.7037	< 2 OK Deep beam				
Third : - Calculate the maximum shear strength of beam cross section : -									
Vu at A = RA =		173.216215	kN						
Vn = 0.83 * sqrt(fc')*(b*d-3.14*50^2/4)		293.856463	kN						
Ø =		0.75							
Ø Vn =		220.3923473	kN	>	173.21622				
Therefore, the cross sectional dimensions are adequate									
Fourth : - Select Strut and Tie model and geometry : -									
For Strut BC									
Fu,BC = Ø Fnc = Ø fce* Ac = Ø*(0.85* βs* fc')b ws				βs =	1	Horizontal strut (C-C-C node)			
For Tie AD									
Fu,AD = Ø Fnt = Ø fce* Ac = Ø*(0.85* βn* fc')b wt				βn =	0.8	(C-C-T node)			
From Model Strut BC and Tie AD form a couple , therefore									
Fu,BC = Fu,AD		OR							
Ø*(0.85* βs* fc')b ws = Ø*(0.85* βn* fc')b wt									
So,	wt = 1.25 ws								
jd = H - wsl2 - wt/2		jd = 405 - ws/2 - wt/2		wt = 1.25 ws		So,			
jd = 405 - 1.125 ws									
Take a moment a bout point (A) we get : -									
Vu* a - Fu,BC* jd = 0				Vu * a - (0.85* βs* fc')b ws * (405 - 1.125 ws)=0					
(-Vu * a) + (0.85* βs* fc')b ws * 405 + (0.85* βs* fc')b ws * 1.125 ws =0					βs =	1			
(-Vu * a) + 289 fc' b ws + 0.95625* fc' b ws^2 =0			0.95625* fc' b ws^2 + 289 fc' b ws -Vu * a =0						
Let A =	0.95625* fc' b	19364.0625							
Let B =	289 fc' b	6971062.5	ws 1 =	-376.4	NEGLECT				
Let C =	-Vu * a	-119519188.4	ws 2 =	16.3981	OK	wt = 1.25 ws			
jd = 405 - 1.125 ws		386.5521291			wt =	20.4976343			
θ = tan⁻¹ (386.356253/ 640)		θ =	29.25846677	>	25°	OK.			386.552
tanθ = (149.216215/Fu,AD)		Fu,AD (Tie) =Fu,BC (Strut)	309.2624914	k N					
Fu,AD (Tie) =		Ø As * fy		As = Fu,AD / Ø * fy				690	
As =	981.7856871	mm²							
Avh = Av	0.0025 *b*S2	S2 = d/5	OR	300					
S2 = d/5		67.4	<	300	So, use it				
Avh1= Av	25.275	Use Ø 6		Area of bar (6) =		28.2735			386.552
Avh2=Av	56.547	Avh2= 2.1 Avh1 so must increase S		S= (d/5)*2	134.8				
			405-(20*2)-(8*2)-25.4 =		323.6	mm	Fu,AD =		
	No. of bars for Avh =	2.400593472	So use 3 bars in each side				Fu,BC		
Finaly					Whith space S	107.866667			

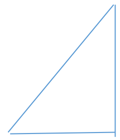
As =	*981.7856871*	*so use 2 Φ 25 then As =*	*981.7856871*	*mm²*	
Avh =Av	*56.547*	*Use 6 Ø 6 @ 46.86 mm*	*113.094*	*mm²*	*4 bar Ø 6 mm because two already excisting to form the steel cage*
		As (Total) =	*1094.879687*	*mm²*	



Deep Beam Method				UHPCTB 2				
H t =		315	mm					
H1 =		180	mm					
dc =		216.225	mm					
d2 =		247		Cover	20	mm		
H 2 =		135	mm	bar =	25	mm		
Load gap =		0.3	m	bar 2 =	8	mm		
a =		0.64	mm					
b =		150	mm	dc=ds*F				
L t =		1.9	m	ds=180-(20+8+25/2) =				139.5
Overhange length (m) =		0.16	m					
Ln =		1.58	m	F=(1-3.04 tan)^(-0.608)<= 1.55				3.38383
fc' =		135	Mpa	Use F= 1.55				1.55
fy =		420	Mpa	dc =				216.225
Applied load =		185	kN					
P L = Applied load =		185	kN	Theoritecal			Two point loads	
First : - Calculate the factored load: -				FLAXURAL Failure's load =			298.1	kN
P D =		1.749735	kN	Maximum Capasity of testing device =			600	kN
Pu =		298.099682	kN	Experemental SHEAR failure's load			362	kN
R A = R B =		149.049841	kN					
Second: - Check if the beam is deep								
Clear span , Ln =	1.58	(Ln/H)*1000	5.01587	> 4 Not Deep beam				
Shear span , a =	0.64	(a/H)*1000	2.03175	> 2 Not Deep beam				

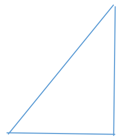
Deep Beam Method				UHPCTB 3				
H t =		360	mm					
H1 =		180	mm					
dc =		216.225	mm					
d2 =		292		Cover	20	mm		
H 2 =		180	mm	bar =	25	mm		
Load gap =		0.3	m	bar 2 =	8	mm		
a =		0.64	mm					
b =		150	mm	dc=ds*F				
L t =		1.9	m	ds=180-(20+8+25/2) =				139.5
Overhange length (m) =		0.16	m					
Ln =		1.58	m	F=(1-3.04 tan)^(-0.608) <= 1.55				3.38383
fc' =		135	Mpa	Use F= 1.55				1.55
fy =		420	Mpa	dc =				216.225
Applied load =		204	kN					
P L = Applied load =		204	kN				Two point loads	
First : - Calculate the factored load: -				Theoritecal FLAXURAL Failure's load =			328.666	kN
P D =		1.88838	kN	Maximum Capasity of testing device =			600	kN
Pu =		328.666056	kN	Experemental SHEAR failure's load =			375	kN
R A = R B =		164.333028	kN				1.14098	
Second: - Check if the beam is deep								
Clear span , Ln =	1.58	(Ln/H)*1000	4.388889	> 4 Not Deep beam				
Shear span , a =	0.64	(a/H)*1000	1.777778	< 2 OK Deep beam				
Third : - Calculate the maximum shear strength of beam cross section : -								
Vu at A = RA =			164.333	kN				
Vn = 0.83 * sqrt(fc')*(b*d-3.14*50^2/4)			293.8565	kN				
Ø =	0.75							
Ø Vn =	220.3923473	kN	>	164.333				
Therefore, the cross sectional dimensions are adequate								
Fourth : - Select Strut and Tie model and geometry : -								
For Strut BC								
Fu,BC=ØFnc=Øfce*Ac=Ø*(0.85*βs*fc')b ws			βs =	1	Horizontal strut(C-C-Cnode)			
For Tie AD								
Fu,AD = Ø Fnt = Ø fce* Ac = Ø*(0.85* βn* fc')b wt			βn =	0.8	(C-C-T node)			
From Model Strut BC and Tie AD form a couple , therefore								
Fu,BC = Fu,AD	OR							

Ø*(0.85* βs* fc')b ws = Ø*(0.85* βn* fc')b wt							
So,	wt = 1.25 ws						
jd = H - wsl2 - wt/2		jd = 405 - ws/2 - wt/2		wt = 1.25 ws		So,	
jd = 405 - 1.125 ws							
Take a moment a bout point (A) we get							
Vu* a - Fu,BC* jd = 0			Vu * a - (0.85* βs* fc')b ws * (405 - 1.125 ws)=0				
(-Vu * a) + (0.85* βs* fc')b ws * 405 + (0.85* βs* fc')b ws * 1.125 ws =0					βs =	1	
(-Vu * a) + 289 fc' b ws + 0.95625* fc' b ws^2 =0				0.95625* fc' b ws^2 + 289 fc' b ws -Vu * a =0			
Let A =	0.95625* fc' b	19364.0625					
Let B =	289 fc' b	6196500	ws 1 =	-336.157	NEGLECT		
Let C =	-Vu * a	-105173137.9	ws 2 =	16.15719	OK	wt = 1.25 ws	
jd = 405 - 1.125 ws		341.8231565			wt =	20.1965	
(θ) = $\tan^{-1}$ (386.356253/ 640)		θ =	28.10664007	>	25°	OK.	
tan(θ) = (149.216215/Fu,AD)		Fu,AD(Tie)=Fu,BC(Strut)	307.7511176	k N			
Fu,AD (Tie) =		Ø As * fy		As = Fu,AD / Ø * fy			
As =	976.987675	mm^2					
Avh = Av	0.0025 *b*S2	S2 = d/5	OR	300			
S2 = d/5		58.4	<	300	So, use it		
Avh1= Av	21.9	Use Ø 6		Area of bar (6) =		28.2735	
Avh2= Av	56.547	Avh2=2.1Avh1 so must increase S=		S= (d/5)*2	116.8		
			405-(20*2)-(8*2)-25.4 =		323.6	mm	
			No. of bars for Avh =		2.7705	So use 3 bars in each side	
Finaly				Whith space S		107.867	
As =	976.987675	so use 2 Φ 25 then As =	976.9876749	mm^2			
Avh=Av	56.547	Use 6 Ø 6 @ 107.8 mm	113.094	mm^2	4 bar Ø 6 mm because two already excisting		
		As(Total)=	1090.081675	mm^2			



341.8231565

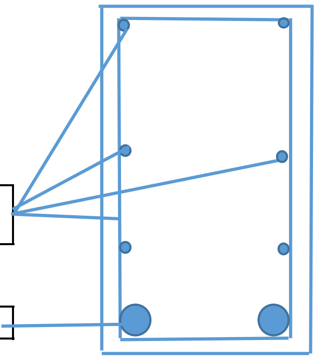
640



341.8231565

Fu,AD

=Fu,BC



Ø 6 @ 107.8 mm

2 Φ 25.4 mm

Deep Beam Method			UHPCTB 5				
H t =	405	mm					
H1 =	180	mm					
d1 =	216.225	mm					
d2 =	337		Cover	20	mm		
H 2 =	225	mm	bar =	25	mm		
Load gap =	0.3	m	bar 2 =	8	mm		
a =	0.64	mm					
b =	150	mm	dc=ds*F				
L t =	1.9	m	ds=180-(20+8+25/2) =				139.5
Overhang length (m) =	0.16	m					
Ln =	1.58	m	F=(1-3.04 tan)^(-0.608) <= 1.55				3.38383
fc' =	135	Mpa	Use F= 1.55				1.55
fy =	420	Mpa	dc =				216.225
Applied load =	231	kN					
P L = Applied load =	231	kN				Two point loads	
First : - Calculate the factored load: -			Theoritecal flexural Failure's load			372.032	kN
P D =	2.027025	kN	Maximum Capasity of testing device			600	kN
Pu =	372.0324	kN	Experemental shear failure's load			462	kN
R A = R B =	186.0162	kN	Highest shear failure load			590	
Second: - Check if the beam is deep						1.58588	

Clear span , Ln =	1.58	(Ln/H)*1000	3.901235	< 4	OK Deep beam
Shear span , a =	0.64	(a/H)*1000	1.580247	< 2	OK Deep beam

Third : - Calculate the maximum shear strength of beam cross section : -

Vu at A = RA =	186.016	kN		
Vn = 0.83 * sqrt(fc')*b*d-(3.14*50^2/4)		293.8565	kN	
Ø =	0.75			
Ø Vn =	220.392	kN	>	186.0162

Therefore, the cross sectional dimensions are adequate

Fourth : - Select Strut and Tie model and geometry : -

For Strut BC			
Fu,BC=ØFnc=Ø fce*Ac=Ø*(0.85* βs* fc')b ws	βs =	1	Horizontal strut (C-C-C node)
For Tie AD			
Fu,AD=ØFnt=Øfce*Ac=Ø(0.85βn*fc')b wt	βn =	0.8	(C-C-T node)

From Model Strut BC and Tie AD form a couple , therefore

Fu,BC = Fu,AD	OR

Ø*(0.85* βs* fc')b ws = Ø*(0.85* βn* fc')b wt

So,	wt = 1.25 ws

jd=H-wsl2-wt/2	jd=405-ws/2-wt/2	wt = 1.25 ws	So,

jd = 405 - 1.125 ws

Take a moment a bout (A) we get

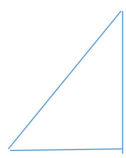
Vu* a - Fu,BC* jd = 0 → Vu * a - (0.85* βs* fc')b ws * (405 - 1.125 ws)=0

(-Vu * a) + (0.85* βs* fc')b ws * 405 + (0.85* βs* fc')b ws * 1.125 ws =0	βs =	1

(-Vu*a)+289fc' b ws+0.95625fc' b ws^2=0	0.95625* fc' b ws^2 + 289 fc' b ws -Vu * a =0

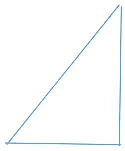
Let A =	0.95625* fc' b			19364.06		
Let B =	289 fc' b	6971063	ws 1 =	-376.3365	NEGLECT	
Let C =	-Vu * a	-1E+08	ws 2 =	16.33646	OK	wt = 1.25 ws
jd = 405 - 1.125 ws		386.621			wt =	20.420577

(θ) = tan⁻¹ (386.356253/ 640)	θ =	31.13604	>	25°	OK.



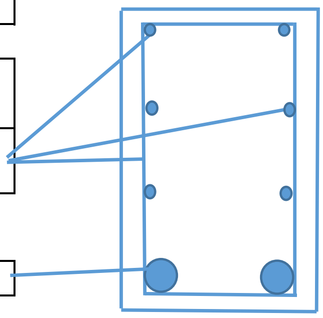
tan(θ)=(149.216215/Fu,AD)	Fu,AD(Tie)=Fu,BC(Strut)	307.9959	k N
Fu,AD (Tie) =	Ø As * fy →	As = Fu,AD / Ø * fy	

As =	977.765	mm²					
Avh = Av	0.0025 *b*S2		S2 = d/5	OR	300		
S2 = d/5		67.4	<	300	So, use it		
Avh1= Av	25.275	Use Ø 6		Area of bar (6) =		28.2735	
Avh2= Av	56.547	Avh2=2.1 Avh1 so must increase S			S= (d/5)*2	134.8	
		405-(20*2)-(8*2)-25.4 =			323.6	mm	Fu,AD = Fu,BC
			No. of bars for Avh =		2.4005935	So use 3 bars in each side	
Finaly					Whith space S	107.86667	
As =	977.765	2Φ25As		977.7648			
Avh = Av	56.547	6Ø6 @107.8		113.094	4 bar Ø 6 mm because two already		
		As (Total) =		1090.859			



Ø 6 @ 107.8 mm

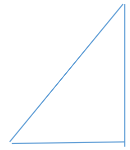
2 Φ 25.4 mm



Deep Beam Method				***UHPCTB 6***				
H t =		***405***	***mm***					
H1 =		***180***	***mm***					
d1 =		***216.225***	***mm***					
d2 =		***337***		***Cover***	***20***	***mm***		
H 2 =		***225***	***mm***	***bar =***	***25***	***mm***		
Load gap =		***0.3***	***m***	***bar 2 =***	***8***	***mm***		
a =		***0.64***	***mm***					
b =		***150***	***mm***	***dc=ds*F***				
L t =		***1.9***	***m***	***ds=180-(20+8+25/2) =***				***139.5***
Overhange length (m) =		***0.16***	***m***					
Ln =		***1.58***	***m***	***F=(1-3.04 tan)^(-0.608) <= 1.55***				***3.38383***
fc' =		***135***	***Mpa***	***Use F= 1.55***				***1.55***
fy =		***420***	***Mpa***	***dc =***				***216.225***
Applied load =		***232***	***kN***					
P L = Applied load =		***232***	***kN***				***Two point loads***	
First : - Calculate the factored load: -				***Theoritecal FLAXURAL Failure's load =***			***373.632***	***kN***
P D =		***2.027025***	***kN***	***Maximum Capasity of testing device =***			***600***	***kN***
Pu =		***373.63243***	***kN***	***Experemental SHEAR failure's load =***			***486***	***kN***
R A = R B =		***186.816215***	***kN***	***Highest shear failure load***			***590***	
Second: - Check if the beam is deep							***1.57909***	
Clear span , Ln =	***1.58***	***(Ln / H) * 1000 =***	***3.90123***	***< 4 OK Deep beam***				
Shear span , a =	***0.64***	***(a / H) *1000 =***	***1.58025***	***< 2 OK Deep beam***				
Third : - Calculate the maximum shear strength of beam cross section : -								
Vu at A = RA =	***186.816215***	***kN***						
Vn = 0.83 * sqrt(fc')*b*d	***312.7822803***	***kN***						
Ø =	***0.75***							
Ø Vn =	***234.5867102***	***kN***	***>***	***186.816***				
Therefore, the cross sectional dimensions are adequate								
Fourth : - Select Strut and Tie model and geometry : -								
For Strut BC								
Fu,BC = Ø Fnc = Ø fce* Ac = Ø*(0.85* βs* fc')b ws			***βs =***	***1***	***Horizontal strut (C-C-C node)***			
For Tie AD								
Fu,AD = Ø Fnt = Ø fce* Ac = Ø*(0.85* βn* fc')b wt			***βn =***	***0.8***	***(C-C-T node)***			

From Model Strut BC and Tie AD form a couple , therefore							
Fu,BC = Fu,AD		OR					
Ø*(0.85* βs* fc')b ws = Ø*(0.85* βn* fc')b wt							
So,	wt = 1.25 ws						
jd = H - wsl2 - wt/2		jd = 405 - ws/2 - wt/2	wt = 1.25 ws		So,		
jd = 405 - 1.125 ws							
Take a moment a bout point (A) we get : -							
Vu* a - Fu,BC* jd = 0				Vu * a - (0.85* βs* fc')b ws * (405 - 1.125 ws)=0			
(-Vu * a) + (0.85* βs* fc')b ws * 405 + (0.85* βs* fc')b ws * 1.125 ws =0					βs =	1	
(-Vu * a) + 289 fc' b ws + 0.95625* fc' b ws^2 =0			0.95625* fc' b ws^2 + 289 fc' b ws -Vu * a =0				
Let A =	0.95625* fc' b	19364.0625					
Let B =	289 fc' b	6971062.5	ws 1 =	-376.4	NEGLECT		
Let C =	-Vu * a	-119562377.6	ws 2 =	16.4038	OK	wt = 1.25 ws	
jd = 405 - 1.125 ws		386.5457412			wt =	20.5047	

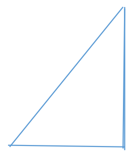
(θ) = tan^{-1} (386.356253/ 640)	θ =	31.13106717	>	25°	OK.	



386.5457412

640

tan(θ) = (149.216215/Fu,AD)		Fu,AD (Tie) =Fu,BC (Strut)	309.3811107	k N			
Fu,AD (Tie) =		Ø As * fy		As = Fu,AD / Ø * fy			
As =	982.162256	mm^2					
Avh = Av	0.0025 *b*S2	S2 = d/5	OR	300			
S2 = d/5		67.4	<	300	So, use it		
Avh1= Av	25.275	Use Ø 6		Area of bar (6) =		28.2735	
Avh2= Av	56.547	Avh2 = 2.1 Avh1 so must increase S =		S= (d/5)*2	134.8		
			405-(20*2)-(8*2)-25.4 =		323.6	mm	
			No. of bars for Avh =		2.40059	So use 3 bars in each side	
					Whith space	107.867	



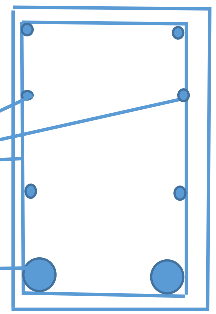
386.5457412

Fu,AD = Fu,BC

Finaly						
As =	982.162256	so use 2 Φ 25 then As =	982.162256	mm^2		
Avh = Av	56.547	Use 6 Ø 6 @ 107.8 mm	113.094	mm^2	4 bar Ø 6 mm	
		As (Total) =	1095.256256	mm^2	because two	

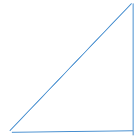
Ø 6 @ 107.8 mm

2 Φ 25.4 mm



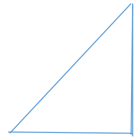
Deep Beam Method				UHPCTB 7				
H t =		405	mm					
H1 =		180	mm					
d shear =		187.55	mm					
d flexural =		346	mm	Cover	20	mm		
H 2 =		225	mm	bar	16	mm		
Load gap =		0.3	m	bar 2 =	8	mm		
a =		0.64	mm					
b =		150	mm	dc=ds*F				
L t =		1.9	m	ds=180-(20+8+16+15) =				121
Overhange length (m) =		0.16	m					
Ln =		1.58	m	F=(1-3.04 tan)^(-0.608) <= 1.55				3.38383
fc' =		135	Mpa	Use F= 1.55				1.55
fy =		420	Mpa	dc =				187.55
Applied load =		150	kN			Two point loads		
P L = Applied load =		150	kN	Theoritecal FLAXURAL Failure's load			242.4324	kN
First : - Calculate the factored load: -								
P D =		2.027025	kN	Experemental SHEAR failure's load			370	kN
Pu =		242.43243	kN				1.526198	
R A = R B =		121.216215	kN					
Second: - Check if the beam is deep								
Clear span , Ln =	1.58	(Ln / H) * 1000	3.901234568					
Shear span , a =	0.64	(a / H) *1000	1.580246914	< 2 OK Deep beam				
Third : - Calculate the maximum shear strength of beam cross section : -								
Vu at A = RA =	121.216215	kN						
Vn = 0.83 * sqrt(fc')*(b*d-3.14*50^2)	252.3763757	kN						
Ø =	0.75							
Ø Vn =	189.2822818	kN	>	121.216215				
Therefore, the cross sectional dimensions are adequate								
Fourth : - Select Strut and Tie model and geometry : -								
For Strut BC								
Fu,BC = Ø Fnc = Ø fce* Ac = Ø*(0.85* βs* fc')b ws		βs =	1	Horizontal strut (C-C-C node)				
For Tie AD								
Fu,AD = Ø Fnt = Ø fce* Ac = Ø*(0.85* βn* fc')b wt		βn =	0.8	(C-C-T node)				
From Model Strut BC and Tie AD form a couple , therefore								
Fu,BC = Fu,AD	OR							
Ø*(0.85* βs* fc')b ws = Ø*(0.85* βn* fc')b wt								
So,	wt = 1.25 ws							
jd = H - wsl2 - wt/2	jd = 405 - ws/2 - wt/2		wt = 1.25 ws		So,			

jd = 405 - 1.125 ws								
Take a moment a bout point (A) we get : -								
Vu* a - Fu,BC* jd = 0			→	Vu * a - (0.85* βs* fc')b ws * (405 - 1.125 ws)=0				
(-Vu * a) + (0.85* βs* fc')b ws * 405 + (0.85* βs* fc')b ws * 1.125 ws =0					βs =	1		
(-Vu * a) + 289 fc' b ws + 0.95625* fc' b ws^2 =0					→	0.95625* fc' b ws^2 + 289 fc' b ws -Vu * a =0		
Let A =	0.95625* fc' b	19364.0625						
Let B =	289 fc' b	6971062.5	ws 1 =	-370.8043682	NEGLECT			
Let C =	-Vu * a	-77578377.6	ws 2 =	10.80436819	OK	wt = 1.25 ws	wt =	13.50546
jd = 405 - 1.125 ws		392.8450858						
(θ) = tan⁻¹ (386.356253/ 640)		θ =	31.54247996	>	25°	OK.		
tan(θ) = (149.216215/Fu,AD)		Fu,AD (Tie) =Fu,BC (Strut)	197.5240972	k N				
Fu,AD (Tie) =		Ø As * fy	→	As = Fu,AD / Ø * fy				
As =	627.060626	mm²						
Avh =Av	0.0025 *b*S2	S2 = d/5	OR	300				
S2 = d/5		69.2	<	300	So, use it			
Avh1 =Av	25.95	Use Ø 6	→	Area of bar (6)		28.2735	mm²	
Avh2 =Av	56.547	Avh2 = 2.1 Avh1 so must increase S =		S= (d/5)*2	138.4	mm		
			405-(20*2)-(8*2)-16-30-12 =		291	mm		
			No. of bars for Avh =		2.102601156	So use 3 bars in each side		
				Whith space S		97		
Finaly								
As =	627.060626	Use 2 Φ 16+ 2Φ12 So As =	627.060626	mm²	4 bar Ø 6 mm because two already excisting to form the steel cage			
Avh =Av	56.547	Use 6 Ø 6 @ 97 mm	113.094	mm²				
		As (Total) =	740.154626	mm²				



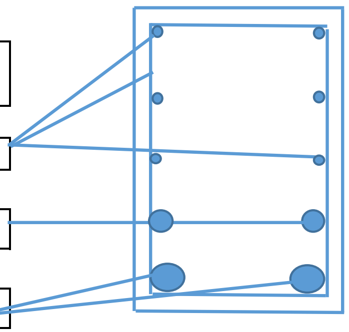
392.8450858

640



392.8450858

Fu,AD = Fu,BC



Ø 6 @ 97 mm

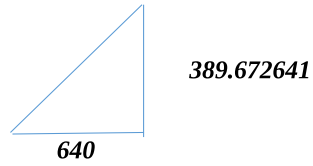
2 Φ 12 mm

2 Φ 16mm

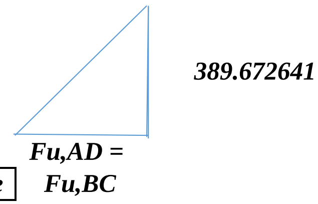
Deep Beam Method				***UHPCTB 8***				
H t =		***405***	***mm***					
H1 =		***180***	***mm***					
d1 =		***187.55***	***mm***					
d2 =		***346***	***mm***	***Cover***	***20***	***mm***		
H 2 =		***225***	***mm***	***bar***	***16***	***mm***		
Load gap =		***0.3***	***m***	***bar 2 =***	***8***	***mm***		
a =		***0.64***	***mm***					
b =		***150***	***mm***	***dc=ds*F***				
L t =		***1.9***	***m***	***ds=180-(20+8+16+15) =***				***121***
Overhange length (m) =		***0.16***	***m***					
Ln =		***1.58***	***m***	***F=(1-3.04 tan)^(-0.608) <= 1.55***				***3.38383***
fc' =		***135***	***Mpa***	***Use F= 1.55***				***1.55***
fy =		***420***	***Mpa***	***dc =***				***187.55***
Applied load =		***191***	***kN***			***Two point loads***		
P L = Applied load =		***191***	***kN***	***Theoritecal FLAXURAL Failure's load***			***308.03243***	***kN***
First : - Calculate the factored load: -								
P D =		***2.027025***	***kN***	***Experemental SHEAR failure's load***			***446***	***kN***
Pu =		***308.03243***	***kN***				***1.4478995***	
R A = R B =		***154.016215***	***kN***					
Second: - Check if the beam is deep								
Clear span , Ln =	***1.58***	***(Ln / H) * 1000***	***3.90123457***	***< 4 OK Deep beam***				
Shear span , a =	***0.64***	***(a / H) *1000***	***1.58024691***	***< 2 OK Deep beam***				
Third : - Calculate the maximum shear strength of beam cross section : -								
Vu at A = RA =	***154.016215***	***kN***						
Vn = 0.83 * sqrt(fc')*(b*d-3.14*50^2/4)	***252.3763757***	***kN***						
Ø =	***0.75***							
Ø Vn =	***189.2822818***	***kN***	***>***	***154.016215***				
Therefore, the cross sectional dimensions are adequate								
Fourth : - Select Strut and Tie model and geometry : -								
For Strut BC								
Fu,BC = Ø Fnc = Ø fce* Ac = Ø*(0.85* βs* fc')b ws		***βs =***	***1***	***Horizontal strut (C-C-C node)***				
For Tie AD								
Fu,AD = Ø Fnt = Ø fce* Ac = Ø*(0.85* βn* fc')b wt		***βn =***	***0.8***	***(C-C-T node)***				
From Model Strut BC and Tie AD form a couple , therefore								
Fu,BC = Fu,AD	***OR***							
Ø*(0.85* βs* fc')b ws = Ø*(0.85* βn* fc')b wt								
So, ***wt = 1.25 ws***								
jd = H - wsl2 - wt/2	***jd = 405 - ws/2 - wt/2***		***wt = 1.25 ws***		***So,***			

jd = 405 - 1.125 ws								
Take a moment a bout point (A) we get : -								
Vu* a - Fu,BC* jd = 0			→	Vu * a - (0.85* βs* fc')b ws * (405 - 1.125 ws)=0				
(-Vu * a) + (0.85* βs* fc')b ws * 405 + (0.85* βs* fc')b ws * 1.125 ws =0					βs =	1		
(-Vu * a) + 289 fc' b ws + 0.95625* fc' b ws^2 =0					→	0.95625* fc' b ws^2 + 289 fc' b ws -Vu * a =0		
Let A =	0.95625* fc' b	19364.0625						
Let B =	289 fc' b	6971062.5	ws 1 =	-373.62432	NEGLECT			
Let C =	-Vu * a	-98570377.6	ws 2 =	13.6243191	OK	wt = 1.25 ws	wt =	17.030399
jd = 405 - 1.125 ws		389.672641						

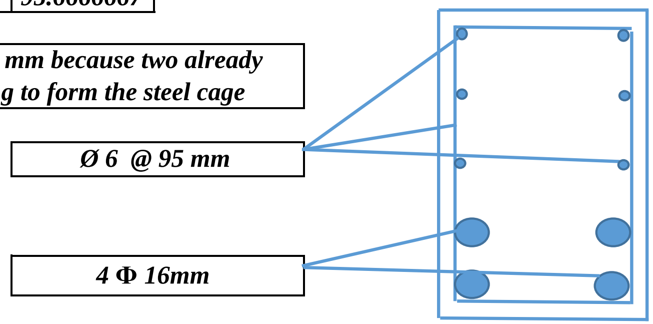
(θ) = tan⁻¹ (386.356253/ 640)	θ =	31.33573595	>	25°	OK.



tan(θ) = (149.216215/Fu,AD)		Fu,AD (Tie) =Fu,BC (Strut)	253.0153868	k N			
Fu,AD (Tie) =		Ø As * fy	→	As = Fu,AD / Ø * fy			
As =	803.2234503	mm²					
Avh = Av	0.0025 *b*S2	S2 = d/5	OR	300			
S2 = d/5		69.2	<	300	So, use it		
Avh1 =Av	25.95	Use Ø 6	→	Area of bar (6)		28.2735	mm²
Avh2 =Av	56.547	Avh2 = 2.1 Avh1 so must increase S =		S= (d/5)*2	138.4	mm	
			405-(20*2)-(8*2)-16-30-16 =		287	mm	
			No. of bars for Avh =		2.07369942	So use 3 bars in each side	
				Whith space S		95.6666667	



Finaly					
As =	803.2234503	so use 4 Φ 16 then As =	803.2234503	mm²	4 bar Ø 6 mm because two already excisting to form the steel cage
Avh =Av	56.547	Use 6 Ø 6 @ 95 mm	113.094	mm²	
		As (Total) =	916.3174503	mm²	



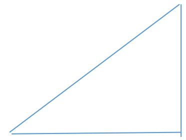
Deep Beam Method				***UHPCTB 9***				
H t =		***405***	***mm***					
H1 =		***180***	***mm***					
d1 =		***216.225***	***mm***					
d2 =		***337***		***Cover***	***20***	***mm***		
H 2 =		***225***	***mm***	***bar =***	***25***	***mm***		
Load gap =		***0.5***	***m***	***bar 2 =***	***8***	***mm***		
a =		***0.54***	***mm***					
b =		***150***	***mm***	***dc=ds*F***				
L t =		***1.9***	***m***	***ds=180-(20+8+25/2) =***				***139.5***
Overhange length (m) =		***0.16***	***m***					
Ln =		***1.58***	***m***	***F=(1-3.04 tan)^(-0.608) <= 1.55***				***3.38383***
fc' =		***135***	***Mpa***	***Use F= 1.55***				***1.55***
fy =		***420***	***Mpa***	***dc =***				***216.225***
Applied load =		***273***	***kN***					
P L = Applied load =		***273***	***kN***				***Two point loads***	
First : - Calculate the factored load: -				***Theoritecal FLAXURAL Failure's load =***			***439.23243***	***kN***
P D =		***2.027025***	***kN***	***Maximum Capasity of testing device =***			***600***	***kN***
Pu =		***439.23243***	***kN***	***Experemental SHEAR failure's load =***			***460***	***kN***
R A = R B =		***219.616215***	***kN***	***Highest shear failure load***			***590***	
Second: - Check if the beam is deep							***1.343252364***	
Clear span , Ln =	***1.58***	***(Ln/H)*1000***	***3.90123***	***< 4***	***OK Deep beam***			
Shear span , a =	***0.54***	***(a / H) *1000***	***1.33333***	***< 2***	***OK Deep beam***			
Third : - Calculate the maximum shear strength of beam cross section : -								
Vu at A = RA =	***219.616215***	***kN***						
Vn = 0.83 * sqrt(fc')*(b*d-3.14*50^2/4)	***293.856463***	***kN***						
Ø =	***0.75***							
Ø Vn =	***220.3923473***	***kN***	***>***	***219.616***				
Therefore, the cross sectional dimensions are adequate								
Fourth : - Select Strut and Tie model and geometry : -								
For Strut BC								
Fu,BC = Ø Fnc = Ø fce* Ac = Ø*(0.85* βs* fc')b ws			***βs =***	***1***	***Horizontal strut (C-C-C node)***			
For Tie AD								
Fu,AD = Ø Fnt = Ø fce* Ac = Ø*(0.85* βn* fc')b wt			***βn =***	***0.8***	***(C-C-T node)***			
From Model Strut BC and Tie AD form a couple , therefore								
Fu,BC = Fu,AD	***OR***							
Ø*(0.85* βs* fc')b ws = Ø*(0.85* βn* fc')b wt								
So,	***wt = 1.25 ws***							
jd = H - wsl2 - wt/2	***jd = 405 - ws/2 - wt/2***		***wt = 1.25 ws***		***So,***			

jd = 405 - 1.125 ws						
Take a moment a bout point (A) we get : -						
Vu* a - Fu,BC* jd = 0			Vu * a - (0.85* βs* fc')b ws * (405 - 1.125 ws)=0			
(-Vu * a) + (0.85* βs* fc')b ws * 405 + (0.85* βs* fc')b ws * 1.125 ws =0					βs =	1
(-Vu * a) + 289 fc' b ws + 0.95625* fc' b ws^2 =0			0.95625* fc' b ws^2 + 289 fc' b ws -Vu * a =0			
Let A =	0.95625* fc' b	19364.0625				
Let B =	289 fc' b	6971062.5	ws 1 =	-376.28	NEGLECT	
Let C =	-Vu * a	-118592756.1	ws 2 =	16.2763	OK	wt = 1.25 ws
jd = 405 - 1.125 ws		386.6891974			wt =	20.34533618

(θ) = $\tan^{-1}$ (388.12/ 540)	θ =	35.60614434	>	25°	OK.

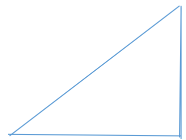
tan(θ) = (149.216215/Fu,AD)		Fu,AD (Tie) =Fu,BC (Strut)	306.7631609	k N		
Fu,AD (Tie) =		Ø As * fy		As = Fu,AD / Ø * fy		
As =	973.8513045	mm²				
Avh =Av	0.0025 *b*S2	S2 = d/5	OR	300		
S2 = d/5		67.4	<	300	So, use it	
Avh1= Av	25.275	Use Ø 6		Area of bar (6) =		28.2735
Avh2=Av	56.547	Avh2 = 2.1 Avh1 so must increase S =		S=(d/5)*2	134.8	
		405-(20*2)-(8*2)-25.4 =			323.6	mm
			No. of bars for Avh =		2.40059	So use 3 bars in each side

Finaly				Whith space S		107.8666667
As =	973.8513045	so use 2 Φ 25 then As =	973.8513045	mm²		
Avh =Av	56.547	Use 6 Ø 6 @ 107 mm	113.094	mm²	4 bar Ø 6 mm because two already excisting to form	
		As (Total) =	1086.945305	mm²		



386.6891974

540

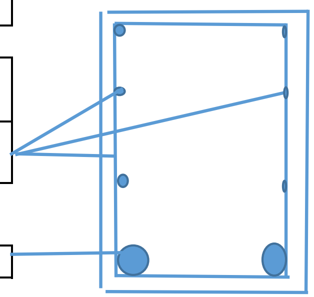


386.6891974

Fu,AD = Fu,BC

Ø 6 @ 107 mm

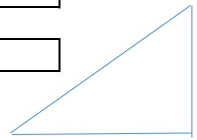
2 Φ 25.4 mm



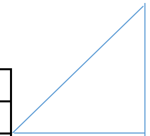
Deep Beam Method				UHPCTB 10				
H t =		405	mm					
H1 =		180	mm					
d1 =		216.225	mm					
d2 =		337		Cover	20	mm		
H 2 =		225	mm	bar =	25	mm		
Load gap =		0.3	m	bar 2 =	8	mm		
a =		0.64	mm					
b =		150	mm	dc=ds*F				
L t =		1.9	m	ds=180-(20+8+25/2) =				139.5
Overhange length (m) =		0.16	m					
Ln =		1.58	m	F=(1-3.04 tan)^(-0.608) <= 1.55				3.38383
fc' =		135	Mpa	Use F= 1.55				1.55
fy =		420	Mpa	dc =				216.225
Applied load =		231	kN					
P L = Applied load =		231	kN				Two point loads	
First : - Calculate the factored load: -				Theoritecal FLAXURAL Failure's load =			372.03243	kN
P D =		2.027025	kN	Maximum Capasity of testing device =			600	kN
Pu =		372.03243	kN	Experemental SHEAR failure's load =			432	kN
R A = R B =		186.016215	kN	Highest shear failure load			590	
Second: - Check if the beam is deep							1.585883252	
Clear span , Ln =	1.58	(Ln / H) * 1000 =	3.901235	< 4 OK Deep beam				
Shear span , a =	0.64	(a / H) *1000 =	1.580247	< 2 OK Deep beam				
Third : - Calculate the maximum shear strength of beam cross section : -								
Vu at A = RA =	186.016215	kN						
Vn = 0.83 * sqrt(fc')*b*d-(3.14*50^2/4)	293.856463	kN						
Ø =	0.75							
Ø Vn =	220.3923473	kN	>	186.016215				
Therefore, the cross sectional dimensions are adequate								
Fourth : - Select Strut and Tie model and geometry : -								
For Strut BC								
Fu,BC = Ø Fnc = Ø fce* Ac = Ø*(0.85* βs* fc')b ws			βs =	1	Horizontal strut (C-C-C node)			
For Tie AD								
Fu,AD = Ø Fnt = Ø fce* Ac = Ø*(0.85* βn* fc')b wt			βn =	0.8	(C-C-T node)			
From Model Strut BC and Tie AD form a couple , therefore								
Fu,BC = Fu,AD	OR							
Ø*(0.85* βs* fc')b ws = Ø*(0.85* βn* fc')b wt								
So,	wt = 1.25 ws							
jd = H - wsl2 - wt/2	jd = 405 - ws/2 - wt/2		wt = 1.25 ws		So,			

jd = 405 - 1.125 ws							
Take a moment a bout point (A) we get : -							
Vu* a - Fu,BC* jd = 0				Vu * a - (0.85* βs* fc')b ws * (405 - 1.125 ws)=0			
(-Vu * a) + (0.85* βs* fc')b ws * 405 + (0.85* βs* fc')b ws * 1.125 ws =0					βs =	1	
(-Vu * a) + 289 fc' b ws + 0.95625* fc' b ws^2 =0			0.95625* fc' b ws^2 + 289 fc' b ws -Vu * a =0				
Let A =	0.95625* fc' b	19364.0625					
Let B =	289 fc' b	6971062.5	ws 1 =	-376.3365	NEGLECT		
Let C =	-Vu * a	-119050377.6	ws 2 =	16.33646	OK	wt = 1.25 ws	
jd = 405 - 1.125 ws		386.6214804			wt =	20.42057736	

(θ) = $\tan^{-1}$ (386.356253/ 640)	θ =	31.1360351	>	25°	OK.	

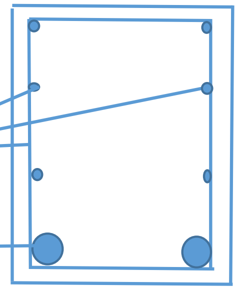


tan(θ) = (149.216215/Fu,AD)		Fu,AD (Tie) =Fu,BC (Strut)	307.9959096	k N			
Fu,AD (Tie) =		Ø As * fy		As = Fu,AD / Ø * fy			
As =	977.7647925	mm^2					
Avh = Av	0.0025 *b*S2	S2 = d/5	OR	300			
S2 = d/5		67.4	<	300	So, use it		
Avh1= Av	25.275	Use Ø 6		Area of bar (6) =		28.2735	
Avh2= Av	56.547	Avh2 = 2.1 Avh1 so must increase S =		S= (d/5)*2	134.8		
			405-(20*2)-(8*2)-25.4 =		323.6	mm	Fu,AD = Fu,BC
			No. of bars for Avh =		2.400593472	So use 3 bars in each side	
Finaly					Whith space S	107.8666667	
As =	977.7647925	so use 2 Φ 25 then As =	977.7647925	mm^2			
Avh = Av	56.547	Use 6 Ø 6 @ 107.8 mm	113.094	mm^2	4 bar Ø 6 mm because two already excisting to form the steel cage		
		As (Total) =	1090.858792	mm^2			



Ø 6 @ 107.8 mm

2 Φ 25 mm



Deep Beam Method		
H t =	405	mm
H1 =	180	mm
d1 =	216.225	mm
d2 =	337	
H 2 =	225	mm
Load gap =	0.3	m
a =	0.64	mm
b =	150	mm
L t =	1.9	m
Overhange length (m) =	0.16	m
Ln =	1.58	m
fc' =	135	Mpa
fy =	420	Mpa
Applied load =	230	kN
P L = Applied load =	230	kN
First : - Calculate the factored load: -		
P D =	2.027025	kN
Pu =	370.43243	kN
R A = R B =	185.216215	kN
Second: - Check if the beam is deep		

UHPCTB 17 + 18

Cover	20	mm
bar =	25	mm
bar 2 =	8	mm

dc=ds*F	
ds=180-(20+8+25.4/2) =	139.5
F=(1-3.04 tan)^(-0.608) <= 1.55	3.38383
Use F= 1.55	1.55
dc =	216.225

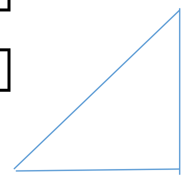
	Two point loads	
Theoritecal FLAXURAL Failure's load =	370.43	kN
Maximum Capasity of testing device =	600	kN
Experemental SHEAR failure's load =	565	kN
Highest shear failure load	590	
	1.5927	

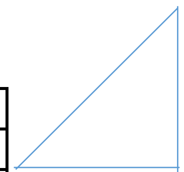
Clear span , Ln =	1.58	(Ln/H)*1000	3.901235	< 4 OK Deep beam
Shear span , a =	0.64	(a/H)*1000	1.580247	< 2 OK Deep beam

Third : - Calculate the maximum shear strength of beam cross section : -

Vu at A = RA =	185.216215	kN		
Vn = 0.83 * sqrt(fc')*(b*d-3.14*50^2/4)	293.856463	kN		
Ø =	0.75			
Ø Vn =	220.3923473	kN	>	185.216215

Therefore, the cross sectional dimensions are adequate							
Fourth : - Select Strut and Tie model and geometry : -							
For Strut BC							
Fu,BC = Ø Fnc = Ø fce Ac = Ø*(0.85* βs* fc')b ws*				*βs =*	*1*	*Horizontal strut (C-C-C node)*	
For Tie AD							
Fu,AD = Ø Fnt = Ø fce Ac = Ø*(0.85* βn* fc')b wt*				*βn =*	*0.8*	*(C-C-T node)*	
From Model Strut BC and Tie AD form a couple , therefore							
Fu,BC = Fu,AD		*OR*					
Ø(0.85* βs* fc')b ws = Ø*(0.85* βn* fc')b wt*							
So,	*wt = 1.25 ws*						
jd = H - wsl2 - wt/2		*jd = 405 - ws/2 - wt/2*		*wt = 1.25 ws*		*So,*	
jd = 405 - 1.125 ws							
Take a moment a bout point (A) we get : -							
Vu a - Fu,BC* jd = 0*				*Vu * a - (0.85* βs* fc')b ws * (405 - 1.125 ws)=0*			
*(-Vu * a) + (0.85* βs* fc')b ws * 405 + (0.85* βs* fc')b ws * 1.125 ws =0*					*βs =*	*1*	
*(-Vu * a) + 289 fc' b ws + 0.95625* fc' b ws^2 =0*			*0.95625* fc' b ws^2 + 289 fc' b ws -Vu * a =0*				
Let A =	*0.95625* fc' b*	*19364.0625*					
Let B =	*289 fc' b*	*6971062.5*	*ws 1 =*	*-376.2691*	*NEGLECT*		
Let C =	*-Vu * a*	*-118538377.6*	*ws 2 =*	*16.26912*	*OK*	*wt = 1.25 ws*	
jd = 405 - 1.125 ws		*386.6972455*			*wt =*	*20.33639*	
(θ) = tan^{-1} (386.356253/ 640)		*θ =*	*31.14100421*	*>*	*25°*	*OK.*	
tan(θ) = (149.216215/Fu,AD)		*Fu,AD(Tie)=Fu,BC (Strut)*	*306.6112306*	*k N*			
Fu,AD (Tie) =		*Ø As * fy*		*As = Fu,AD / Ø * fy*			
As =	*973.368986*	*mm²*					
Avh = Av	*0.0025 *b*S2*	*S2 = d/5*	*OR*	*300*			
S2 = d/5		*67.4*	*<*	*300*	*So, use it*		
Avh1= Av	*25.275*	*Use Ø 6*		*Area of bar (6) =*		*28.2735*	
Avh2= Av	*56.547*	*Avh2 = 2.1 Avh1 so must increase S*		*S= (d/5)*2*	*134.8*		
			*405-(20*2)-(8*2)-25.4 =*		*323.6*	*mm*	*Fu,AD = Fu,BC*



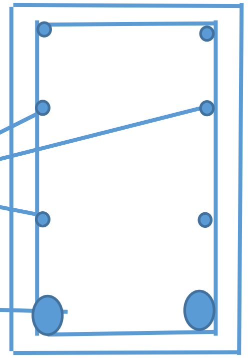


		No. of bars for Avh =	2.400593472	So use 3 bars in each side
			Whith space S	107.8667

Finaly					
As =	973.368986	so use 2 Φ 25 then As =	973.368986	mm^2	
Avh = Av	56.547	Use 6 Ø 6 @ 107.8 mm	113.094	mm^2	4 bar Ø 6 mm because two already excisting to
		As (Total) =	1086.462986	mm^2	

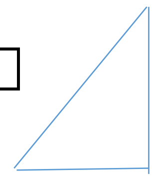
Ø 6 @ 107.8 mm

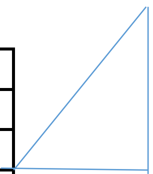
2 Φ 25.4 mm



Deep Beam Method				***UHPCTB 19+20***				
H t =		***405***	***mm***					
H1 =		***180***	***mm***					
d1 =		***187.55***	***mm***					
d2 =		***346***	***mm***	***Cover***	***20***	***mm***		
H 2 =		***225***	***mm***	***bar***	***16***	***mm***		
Load gap =		***0.3***	***m***	***bar 2 =***	***8***	***mm***		
a =		***0.64***	***mm***					
b =		***150***	***mm***	***dc=ds*F***				
L t =		***1.9***	***m***	***ds=180-(20+8+16+15) =***				***121***
Overhange length (m) =		***0.16***	***m***					
Ln =		***1.58***	***m***	***F=(1-3.04 tan)^(-0.608) <= 1.55***				***3.38383***
fc' =		***135***	***Mpa***	***Use F= 1.55***				***1.55***
fy =		***420***	***Mpa***	***dc =***				***187.55***
Applied load =		***191***	***kN***			***Two point loads***		
P L = Applied load =		***191***	***kN***	***Theoritecal FLAXURAL Failure's load***			***308.032***	***kN***
First : - Calculate the factored load: -								
P D =		***2.027025***	***kN***	***Experemental SHEAR failure's load***			***446***	***kN***
Pu =		***308.03243***	***kN***				***1.4479***	
R A = R B =		***154.016215***	***kN***					
Second: - Check if the beam is deep								
Clear span , Ln =	***1.58***	***(Ln/H)*1000***	***3.9012346***	***< 4 OK Deep beam***				
Shear span , a =	***0.64***	***(a/H)*1000***	***1.5802469***	***< 2 OK Deep beam***				
Third : - Calculate the maximum shear strength of beam cross section : -								
Vu at A = RA =	***154.016215***	***kN***						
Vn = 0.83 * sqrt(fc')*b*d-(3.14*50^2/4)	***252.3763757***	***kN***						
Ø =	***0.75***							
Ø Vn =	***189.2822818***	***kN***	***>***	***154.016***				
Therefore, the cross sectional dimensions are adequate								

Fourth : - Select Strut and Tie model and geometry : -								
For Strut BC								
Fu,BC = Ø Fnc = Ø fce* Ac = Ø*(0.85*βs*fc')b ws			βs =	1	Horizontal strut (C-C-C node)			
For Tie AD								
Fu,AD = Ø Fnt = Ø fce* Ac = Ø*(0.85* βn* fc')b wt			βn =	0.8	(C-C-T node)			
From Model Strut BC and Tie AD form a couple , therefore								
Fu,BC = Fu,AD		OR						
Ø*(0.85* βs* fc')b ws = Ø*(0.85* βn* fc')b wt								
So,	wt = 1.25 ws							
jd = H - wsl2 - wt/2		jd = 405 - ws/2 - wt/2		wt = 1.25 ws		So,		
jd = 405 - 1.125 ws								
Take a moment a bout point (A) we get : -								
Vu* a - Fu,BC* jd = 0				Vu * a - (0.85* βs* fc')b ws * (405 - 1.125 ws)=0				
(-Vu * a) + (0.85* βs* fc')b ws * 405 + (0.85* βs* fc')b ws * 1.125 ws =0					βs =	1		
(-Vu * a) + 289 fc' b ws + 0.95625* fc' b ws^2 =0				0.95625* fc' b ws^2 + 289 fc' b ws -Vu * a =0				
Let A =	0.95625* fc' b	19364.0625						
Let B =	289 fc' b	6971062.5	ws 1 =	-373.6243	NEGLECT			
Let C =	-Vu * a	-98570377.6	ws 2 =	13.624319	OK	wt=1.25ws	wt =	17.0304
jd = 405 - 1.125 ws		389.672641						
(θ) = $\tan^{-1}$ (386.356253/ 640)		θ =	31.33573595	>	25°	OK.		
tan(θ) = (149.216215/Fu,AD) Fu,AD (Tie) =Fu,BC (Strut)			253.0153868	k N				
Fu,AD (Tie) =		Ø As * fy		As = Fu,AD / Ø * fy				
As =	803.2234503	mm^2						
Avh = Av	0.0025 *b*S2	S2 = d/5	OR	300				
S2 = d/5		69.2	<	300	So, use it			
Avh1 =Av	25.95	Use Ø 6		Area of bar (6)		28.2735		
Avh2 =Av	56.547	Avh2=2.1 Avh1 so must increase S		S= (d/5)*2	138.4	mm		
			405-(20*2)-(8*2)-16-30-16 =		287	mm	Fu,AD = Fu,BC	



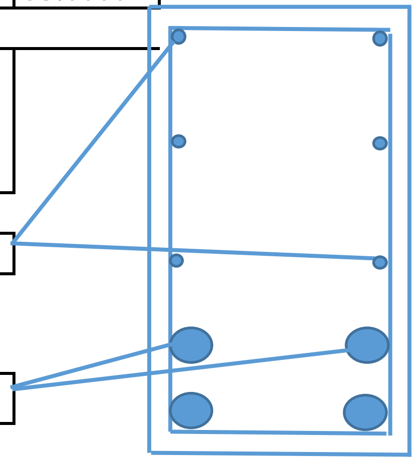


No. of bars for Avh =	***2.0737***	***So use 3 bars in each side***
Whith space S	***95.66667***	

Finaly				
As =	***803.2234503***	***so use 4 Φ 16 then As =***	***803.2234503***	***4 bar Ø 6 because two already excisting to form the steel cage***
Avh =Av	***56.547***	***Use 6 Ø 6 @ 95 mm***	***113.094***	
		As (Total) =	***916.3174503***	

Ø 6 @ 95 mm

4 Φ 16mm



UHPCTB 1

Cover =	20	mm
bar =	25	mm
bar 2 =	8	mm

	Two point loads	
Theoritecal FLAXURAL Failure's load	348	kN
Experemental SHEAR failure's load	416	kN
Highest shear failure load	590	kN
	1.6954	

Irregular shape beam Method				
Ec =	50000	MPa		
H t =	405	mm		
H1 =	180	mm		
H 2 =	225	mm		
d =	364.5	mm		
b =	150	mm		
L t =	1.9	m		
Overhange length (m)	0.16	m		
Ln =	1.58	m		
Load gap =	0.2	m		
shear span (a) =	0.69	m		
fc' =	135	Mpa		
fy =	420	Mpa		
Maximum L.L	348	kN		
wD =	1.1994231	kN/m		
PD =	0.7591285			
Mu =	120.3219	kN		
Assume Z =	328.05	mm		
Ø =	0.9	Factor		
As =	970.31521	mm²		
α =	0.65			
εcu =	0.0035			
Es =	200000	MPa		
β =	0.65			
Find the value of (c) from C = T				
C = (α.fc'.b -0.5*(α.fc')^2 .b/(Ec* εcu) + 0.4*√fc'.b)*(c)^2				
T = (- 0.4*√fc'.b.H + As.Es.εcu) * (c) - As.Es.εcu.d				
A =	(α.fc' b-0.5*b(α.fc')^2 /(Ec* εcu) + 0.4*b*√fc')			13859.637
B =	(- 0.4*√fc'.b.H + As.Es.εcu)			396880.16
D =	- As.Es.εcu.d			-2.48E+08
c 1 =	-148.73543	Neglect		
c 2 =	120.09975	OK		
a =	78.064839	mm		
y° =	a / 2	rectangulare shape		
y° =	39.032419			
Z =	d - y°			
Z =	325.46758	mm	<	328.05
Cycle 2, Recycle with z	325.46758	mm		
As =	978.01416	mm²		
A =	(α.fc' -0.5*(α.fc')^2 /(Ec* εcu) + 0.4*√fc')			13859.637
B =	(- 0.4*√fc'.b.H + As.Es.εcu)			402269.43
D =	- As.Es.εcu.d			-2.5E+08
c 1 =	-149.47681	Neglect		
c 2 =	120.45228	OK		
a =	78.293984			
y° =	a / 2			
y° =	39.146992			
Z =	d - y°			
Z =	325.35301	mm	<	325.46758
Cycle 3, Recycle with z	325.35301	mm		
As =	978.35857	mm²		
A =	(α.fc' -0.5*(α.fc')^2 /(Ec* εcu) + 0.4*√fc')			13859.637
B =	(- 0.4*√fc'.b.H + As.Es.εcu)			402510.51
D =	- As.Es.εcu.d			-2.5E+08
c 1 =	-149.50993	Neglect		
c 2 =	120.46801	OK		
a =	78.304205	mm		
y° =	a / 2			
y° =	39.152103	mm		
Z =	d - y°			

Z =	*325.3479*	*mm*	*Its almost equal to*			*325.353*	*OK*
As(min) 1 =	*(1.4/fy) * (Ac of depth = d)*			*182.25*			
As(min) 2 =	*(0.25* √fc' / fy) * (Ac of depth = d)*			*378.13458*			
As =	*978.35857*	*>*	*As(min) 1 & As(min) 2*		*OK*	*Use As=*	*981.7*
c =	*120.46801*						*mm^2*
εt =	*0.0060771*						
Ø = 0.9 as assumed we have to check it							
Ø	*0.75 + 0.15 * (εt - εy) / (0.005 - εy)*						
Ø	*0.9538549*						
check Ø Mn	*Ø As * fy * (d - y°)*						
Ø Mn	*127.95534*	*>*	*Mu*	*120.3219*	*Ok*		

Irregular shape beam Method			*UHPCTB 2*				
Ec =	*50000*	*MPa*					
H t =	*315*	*mm*					
H1 =	*180*	*mm*	*Cover =*	*10*	*mm*		
H 2 =	*135*	*mm*	*bar =*	*25*	*mm*		
d =	*284.5*	*mm*	*bar 2 =*	*8*	*mm*	*To forming of steel cage*	
b =	*150*	*mm*					
L t =	*1.9*	*m*					
Overhange length (m)	*0.16*	*m*					
Ln =	*1.58*	*m*					
Load gap =	*0.3*	*m*					
shear span (a) =	*0.64*	*m*					
fc' =	*135*	*Mpa*					
fy =	*420*	*Mpa*					
Maximum L.L	*275*	*kN*	*Theoritecal FLAXURAL Failure's load*			*Two point loads*	
wD =	*1.03535*	*kN/m*				*275*	*kN*
PD =	*0.65528*		*Experemental SHEAR failure's load*			*362*	*kN*
Mu =	*88.2097*	*kN*	*Highest shear failure load*			*590*	*kN*
Assume Z =	*256.05*	*mm*				*2.14545*	
Ø =	*0.9*	*Factor*					
As =	*911.38*	*mm^2*					
α =	*0.65*						
εcu =	*0.0035*						
Es =	*200000*	*MPa*					
β =	*0.65*						
Find the value of (c) from C = T							

C = (α.fc'.b -0.5(α.fc')^2 .b/(Ec* εcu) + 0.4*√fc'.b)*(c)^2*		
T = (- 0.4√fc'.b.H + As.Es.εcu) * (c) - As.Es.εcu.d*		
A =	*(α.fc' b-0.5*b(α.fc')^2 /(Ec* εcu) + 0.4*b*√fc')*	*10597.3*
B =	*(- 0.4*√fc'.b.H + As.Es.εcu)*	*418368*
D =	*- As.Es.εcu.d*	*-2E+08*

c 1 =	*-152.09*	*Neglect*		
c 2 =	*112.612*	*OK*		
a =	*73.1975*	*mm*		
y° =	*a / 2*	*rectangulare shape*		
y° =	*36.5987*			
Z =	*d - y°*			
Z =	*247.901*	*mm*	*<*	*256.05*
Cycle 2, Recycle with z	*247.901*	*mm*		
As =	*941.338*	*mm^2*		

A =	*(α.fc' b-0.5*b(α.fc')^2 /(Ec* εcu) + 0.4*b*√fc')*	*10597.3*
B =	*(- 0.4*√fc'.b.H + As.Es.εcu)*	*439339*
D =	*- As.Es.εcu.d*	*-2E+08*

c 1 =	*-155.34*	*Neglect*		
c 2 =	*113.881*	*OK*		
a =	*74.0226*			
y° =	*a / 2*			
y° =	*37.0113*			
Z =	*d - y°*			
Z =	*247.489*	*mm*	*<*	*247.901*
Cycle 3, Recycle with z	*247.489*	*mm*		
As =	*942.908*	*mm^2*		

A =	***(α.fc' b-0.5*b(α.fc')^2 /(Ec* εcu) + 0.4*b*√fc')***					***10597.32014***			
B =	***(- 0.4*√fc'.b.H + As.Es.εcu)***					***440437.1537***			
D =	***- As.Es.εcu.d***					***-187780045.5***			
c 1 =		***-155.51***	***Neglect***						
c 2 =		***113.947***	***OK***						
a =		***74.0653***	***mm***						
y° =		***a / 2***							
y° =		***37.0326***	***mm***						
Z =		***d - y°***							
Z =		***247.467***	***mm***	***Its almost equal to***				***247.489***	***OK***
As(min) 1 =		***(1.4/fy) * (Ac of depth = d)***				***142.25***			
As(min) 2 =		***(0.25* √fc' / fy) * (Ac of depth = d)***				***295.14***			
As =		***942.908***	***>***	***As(min) 1 & As(min) 2***				***Use As=***	***981.7***
c =		***113.947***							***mm²***
εt =		***0.00449***							
Ø = 0.9 as assumed we have to check it									
Ø		***0.75 + 0.15 * (εt - εy) / (0.005 - εy)***							
Ø		***0.87452***							
check Ø Mn		***Ø As * fy * (d - y°)***							
Ø Mn		***89.2308***	***>***	***Mu***	***88.2097***	***Ok***			

Irregular shape beam Method			UHPCTB 3				
Ec =	50000	MPa					
H t =	360	mm					
H1 =	180	mm	Cover =	20	mm		
H 2 =	180	mm	bar =	25	mm		
d =	319.5	mm	bar 2 =	8	mm	To forming of steel	
b =	150	mm					
L t =	1.9	m					
Overhange length (m)	0.16	m					
Ln =	1.58	m					
Load gap =	0.3	m					
shear span (a) =	0.64	m					
fc' =	135	Mpa					
fy =	420	Mpa					
Maximum L.L	322	kN	Theoritecal FLAXURAL			Two point loads	
wD =	1.11738	kN/m	Failure's load			322	kN
PD =	0.70721		Experemental shear failure's load			375	kN
Mu =	103.266	kN	Highest shear failure load			590	kN
Assume Z =	287.55	mm				1.8323	
Ø =	0.9	Factor					
As =	950.065	mm^2					
α =	0.65						
εcu =	0.0035						
Es =	200000	MPa					
β =	0.65						
Find the value of (c) from C = T							
C = (α.fc'.b -0.5*(α.fc')^2 .b/(Ec* εcu) + 0.4*√fc'.b)*(c)^2							
T = (- 0.4*√fc'.b.H + As.Es.εcu) * (c) - As.Es.εcu.d							
A =	(α.fc' b-0.5*b(α.fc')^2 /(Ec* εcu) + 0.4*b*√fc')			10597.3			
B =	(- 0.4*√fc'.b.H + As.Es.εcu)			414076			
D =	- As.Es.εcu.d			-2E+08			
c 1 =	-162.48	Neglect					
c 2 =	123.405	OK					
a =	80.213	mm					
y° =	a / 2	rectangulare shape					
y° =	40.1065						
Z =	d - y°						
Z =	279.394	mm	<	287.55			
Cycle 2, Recycle with z	279.394	mm					
As =	977.801	mm^2					
A =	(α.fc' b-0.5*b(α.fc')^2 /(Ec* εcu) + 0.4*b*√fc')			10597.3			
B =	(- 0.4*√fc'.b.H + As.Es.εcu)			433491			
D =	- As.Es.εcu.d			-2E+08			
c 1 =	-165.55	Neglect					
c 2 =	124.648	OK					
a =	81.0211						
y° =	a / 2						
y° =	40.5105						
Z =	d - y°						
Z =	278.989	mm	<	279.394			
Cycle 3, Recycle with z	278.989	mm					
As =	979.217	mm^2					

A =	***(α.fc' b-0.5*b(α.fc')^2 /(Ec* εcu) + 0.4*b*√fc')***				***10597.32014***			
B =	***(- 0.4*√fc'.b.H + As.Es.εcu)***				***434482.7529***			
D =	***- As.Es.εcu.d***				***-219001937.6***			
c 1 =		***-165.71***	***Neglect***					
c 2 =		***124.711***	***OK***					
a =		***81.0619***	***mm***					
y° =		***a / 2***						
y° =		***40.5309***	***mm***					
Z =		***d - y°***						
Z =		***278.969***	***mm***	***Its almost equal to***			***278.989***	***OK***
As(min) 1 =		***(1.4/fy) * (Ac of depth = d)***				***159.8***		
As(min) 2 =		***(0.25* √fc' / fy) * (Ac of depth = d)***				***331.5***		
As =		***979.217***	***>***	***As(min) 1 & As(min) 2***		***OK***	***Use As=***	***981.7***
c =		***124.711***						***mm²***
εt =		***0.00469***						
Ø = 0.9 as assumed we have to check it								
Ø		***0.75 + 0.15 * (εt - εy) / (0.005 - εy)***						
Ø		***0.88429***						
check Ø Mn		***Ø As * fy * (d - y°)***						
Ø Mn		***101.714***	***>***	***Mu***	***103.266***	***Ok***		

Irregular shape beam Method			UHPCTB 5				
Ec =	50000	MPa					
H t =	405	mm					
H1 =	180	mm	Cover =	20	mm		
H 2 =	225	mm	bar =	25	mm		
d =	364.5	mm	bar 2 =	8	mm	To forming of steel cage	
b =	150	mm					
L t =	1.9	m					
Overhange length (m)	0.16	m					
Ln =	1.58	m					
Load gap =	0.3	m					
shear span (a) =	0.64	m					
fc' =	135	Mpa					
fy =	420	Mpa				Two point loads	
Maximum L.L	370	kN	Theoritecal FLAXURAL Failure's load				
wD =	1.19942	kN/m				370	kN
PD =	0.75913		Experemental SHEAR failure's load			462	kN
Mu =	118.643	kN	Highest shear failure load			590	kN
Assume Z =	328.05	mm				1.59459	
Ø =	0.9	Factor					
As =	956.775	mm^2					
α =	0.65						
εcu =	0.0035						
Es =	200000	MPa					
β =	0.65						
Find the value of (c) from C = T							
C = (α.fc'.b -0.5*(α.fc')^2 .b/(Ec* εcu) + 0.4*√fc'.b)*(c)^2							
T = (- 0.4*√fc'.b.H + As.Es.εcu) * (c) - As.Es.εcu.d							
A =	(α.fc' b-0.5*b(α.fc')^2 /(Ec* εcu) + 0.4*b*√fc')			10597.3			
B =	(- 0.4*√fc'.b.H + As.Es.εcu)			387402			
D =	- As.Es.εcu.d			-2E+08			
c 1 =	-171.15	Neglect					
c 2 =	134.595	OK					
a =	87.4867	mm					
y° =	a / 2	rectangulare shape					
y° =	43.7433						
Z =	d - y°						
Z =	320.757	mm	<	328.05			
Cycle 2, Recycle with z	320.757	mm					
As =	978.53	mm^2					
A =	(α.fc' b-0.5*b(α.fc')^2 /(Ec* εcu) + 0.4*b*√fc')			10597.3			
B =	(- 0.4*√fc'.b.H + As.Es.εcu)			402631			
D =	- As.Es.εcu.d			-2E+08			
c 1 =	-173.66	Neglect					
c 2 =	135.667	OK					
a =	88.1833						
y° =	a / 2						
y° =	44.0917						
Z =	d - y°						
Z =	320.408	mm	<	320.757			

Cycle 3, Recycle with z	***320.408***	***mm***				
As =	***979.594***	***mm^2***				
A =	***(α.fc' b-0.5*b(α.fc')^2 /(Ec* εcu) + 0.4*b*√fc')***	***10597.32014***				
B =	***(- 0.4*√fc'.b.H + As.Es.εcu)***	***403375.5078***				
D =	***- As.Es.εcu.d***	***-249943479.7***				
c 1 =	***-173.78***	***Neglect***				
c 2 =	***135.719***	***OK***				
a =	***88.2171***	***mm***				
y° =	***a / 2***					
y° =	***44.1086***	***mm***				
Z =	***d - y°***					
Z =	***320.391***	***mm***	***Its almost equal to***	***320.408***	***OK***	
As(min) 1 =	***(1.4/fy) * (Ac of depth = d)***	***182.25***				
As(min) 2 =	***(0.25* √fc' / fy) * (Ac of depth = d)***	***378.1345794***				
As =	***979.594***	***>***	***As(min) 1 & As(min) 2***	***OK***	***Use As =***	***981.7***
c =	***135.719***					***mm^2***
εt =	***0.00506***					
Ø = 0.9 as assumed we have to check it						
Ø	***0.75 + 0.15 * (εt - εy) / (0.005 - εy)***					
Ø	***0.90286***					
check Ø Mn	***Ø As * fy * (d - y°)***					
Ø Mn	***119.013***	***>***	***Mu***	***118.643***	***Ok***	

Irregular shape beam Method			*UHPCTB 6*				
Ec =	*50000*	*MPa*					
H t =	*405*	*mm*					
H1 =	*180*	*mm*	*Cover =*	*20*	*mm*		
H 2 =	*225*	*mm*	*bar =*	*25*	*mm*		
d =	*364.5*	*mm*	*bar 2 =*	*8*	*mm*	*To forming of steel*	
b =	*150*	*mm*					
L t =	*1.9*	*m*					
Overhange length (m)	*0.16*	*m*					
Ln =	*1.58*	*m*					
Load gap =	*0.3*	*m*					
shear span (a) =	*0.64*	*m*					
fc' =	*135*	*Mpa*					
fy =	*420*	*Mpa*					
Maximum L.L	*371*	*kN*	*Theoritecal FLAXURAL Failure's load*			*Two point loads*	
wD =	*1.19942308*	*kN/m*				*371*	*kN*
PD =	*0.75912853*		*Experemental shear failure's load*			*486*	*kN*
Mu =	*118.962921*	*kN*	*Highest shear failure load*			*590*	*kN*
Assume Z =	*328.05*	*mm*				*1.59*	
Ø =	*0.9*	*Factor*					
As =	*959.35596*	*mm²*					
α =	*0.65*						
εcu =	*0.0035*						
Es =	*200000*	*MPa*					
β =	*0.65*						
Find the value of (c) from C = T							
C = (α.fc'.b -0.5(α.fc')^2 .b/(Ec* εcu) + 0.4*√fc'.b)*(c)^2*							
T = (- 0.4√fc'.b.H + As.Es.εcu) * (c) - As.Es.εcu.d*							
A =	*(α.fc' b-0.5*b(α.fc')^2 /(Ec* εcu) + 0.4*b*√fc')*			*10597.32014*			
B =	*(- 0.4*√fc'.b.H + As.Es.εcu)*			*459793.8073*			
D =	*- As.Es.εcu.d*			*-244779673.1*			
c 1 =	*-175.21548*	*Neglect*					
c 2 =	*131.827741*	*OK*					
a =	*85.6880319*	*mm*					
y° =	*a / 2*	*rectangulare shape*					
y° =	*42.844016*						
Z =	*d - y°*						
Z =	*321.655984*	*mm*	*<*	*328.05*			
Cycle 2, Recycle with z	*321.655984*	*mm*					
As =	*978.42645*	*mm²*					
A =	*(α.fc' b-0.5*b(α.fc')^2 /(Ec* εcu) + 0.4*b*√fc')*			*10597.32014*			
B =	*(- 0.4*√fc'.b.H + As.Es.εcu)*			*473143.1505*			
D =	*- As.Es.εcu.d*			*-249645508.7*			
c 1 =	*-177.42293*	*Neglect*					
c 2 =	*132.7755*	*OK*					
a =	*86.3040751*						
y° =	*a / 2*						
y° =	*43.1520375*						
Z =	*d - y°*						
Z =	*321.347962*	*mm*	*<*	*321.656*			

Cycle 3, Recycle with z	321.347962	mm				
As =	979.364301	mm^2				
A =	(α.fc' b-0.5*b(α.fc')^2 /(Ec* εcu) + 0.4*b*√fc')	10597.32014				
B =	(- 0.4*√fc'.b.H + As.Es.εcu)	473799.6461				
D =	- As.Es.εcu.d	-249884801.4				
c 1 =	-177.53114	Neglect				
c 2 =	132.821761	OK				
a =	86.3341449	mm				
y° =	a / 2					
y° =	43.1670724	mm				
Z =	d - y°					
Z =	321.332928	mm	Its almost equal to	321.3479625	OK	
As(min) 1 =	(1.4/fy) * (Ac of depth = d)	182.25				
As(min) 2 =	(0.25* √fc' /fy)*(Ac of depth=d)	378.135				
As =	979.364301	>	As(min) 1 & As(min) 2	OK	Use As=	981.7
c =	132.821761					mm^2
εt =	0.00523284					
Ø = 0.9 as assumed we have to check it						
Ø	0.75 + 0.15 * (εt - εy) / (0.005 - εy)					
Ø	0.91164188					
check Ø Mn	Ø As * fy * (d - y°)					
Ø Mn	120.783492	>	Mu	118.963	Ok	

Irregular shape beam Method		
Ec =	50000	MPa
H t =	405	mm
H1 =	180	mm
H 2 =	225	mm
d =	346	mm
b =	150	mm
L t =	1.9	m
Overhange length (m)	0.16	m
Ln =	1.58	m
Load gap =	0.3	m
shear span (a) =	0.64	m
fc' =	135	Mpa
fy =	420	Mpa
Maximum L.L	232	kN
wD =	1.1994231	kN/m
PD =	0.7591285	kN
Mu =	74.482921	kN
Assume Z =	311.4	mm
Ø =	0.9	Factor
As =	632.7706	mm^2
α =	0.65	
εcu =	0.0035	
Es =	200000	MPa
β =	0.65	

UHPCTB 7

The main bars are distrebuted by two rows (2 Φ16 + 2 Φ12)		
Cover =	20	mm
bar =	16	mm
bar 2 =	8	mm

	Two point loads	
Theoritecal FLAXURAL Failure's load =	232	kN
Experemental SHEAR failure's load =	370	kN
	1.59483	

Find the value of (c) from C = T

C = (α.fc'.b -0.5*(α.fc')^2 .b/(Ec* εcu) + 0.4*√fc'.b)*(c)^2

T = (- 0.4*√fc'.b.H + As.Es.εcu) * (c) - As.Es.εcu.d

A =	(α.fc' b-0.5*b(α.fc')^2 /(Ec* εcu) + 0.4*b*√fc')			13859.63694
B =	(- 0.4*√fc'.b.H + As.Es.εcu)			160598.9342
D =	- As.Es.εcu.d			-153257039.4
c 1 =	-111.1093	Neglect		
c 2 =	99.52178	OK		
a =	64.689157	mm		
y° =	a / 2	rectangulare shape		
y° =	32.344578			
Z =	d - y°			
Z =	313.65542	mm	<	311.4
Cycle 2 : - Recycle with z =	313.65542	mm		
As =	628.2205	mm^2		
A =	(α.fc' -0.5*(α.fc')^2 /(Ec* εcu) + 0.4*√fc')			13859.63694
B =	(- 0.4*√fc'.b.H + As.Es.εcu)			157413.8621
D =	- As.Es.εcu.d			-152155004.4
c 1 =	-110.6099	Neglect		
c 2 =	99.252221	OK		
a =	64.513943			
y° =	a / 2			
y° =	32.256972			
Z =	d - y°			
Z =	313.74303	mm	<	313.6554215
Cycle 3 : - Recycle with z =	313.74303	mm		
As =	628.04508	mm^2		
A =	(α.fc' -0.5*(α.fc')^2 /(Ec* εcu) + 0.4*√fc')			13859.63694
B =	(- 0.4*√fc'.b.H + As.Es.εcu)			157291.0691
D =	- As.Es.εcu.d			-152112518
c 1 =	-110.5907	Neglect		
c 2 =	99.241803	OK		
a =	64.507172	mm		
y° =	a / 2			

y° =	*32.253586*	*mm*					
Z =	*d - y°*						
Z =	*313.74641*	*mm*	*Its equal to what we assumed at first*			*313.743*	*OK*
As(min) 1 =	*(1.4/fy) * (Ac of depth = d)*			*173*			
As(min) 2 =	*(0.25* √fc' / fy) * (Ac of depth = d)*			*358.9425637*			
As =	*628.04508*	*>*	*As(min) 1 & As(min) 2*		*OK*	*Use As=*	*628.3*
c =	*99.241803*						*mm²*
εt =	*0.0074593*						
Ø = 0.9 as assumed we have to check it							
Ø	*0.75 + 0.15 * (εt - εy) / (0.005 - εy)*						
Ø	*1.0229651*						
check Ø Mn	*Ø As * fy * (d - y°)*						
Ø Mn	*84.694643*	*>*	*Mu*	*74.48292113*	*Ok*		

Irregular shape beam Method		
Ec =	50000	MPa
H t =	405	mm
H1 =	180	mm
H 2 =	225	mm
d =	346	mm
b =	150	mm
L t =	1.9	m
Overhange length (m)	0.16	m
Ln =	1.58	m
Load gap =	0.3	m
shear span (a) =	0.64	m
fc' =	135	Mpa
fy =	420	Mpa
Maximum L.L	329	kN
wD =	1.199423077	kN/m
PD =	0.75912853	
Mu =	105.5229211	kN
Assume Z =	311.4	mm
Ø =	0.9	Factor
As =	896.4713135	mm^2
α =	0.65	
εcu =	0.0035	
Es =	200000	MPa
β =	0.65	

UHPCTB 8 +20

The main bars are distrebuted by two rows 4 Φ16		
Cover =	20	mm
bar =	16	mm
bar2 =	8	mm

	Two point loads	
Theoritecal FLAXURAL Failure's load	329	kN
Experemental SHEAR failure's load	446	kN
	1.35562	

Find the value of (c) from C = T

C = (α.fc'.b -0.5*(α.fc')^2 .b/(Ec* εcu) + 0.4*√fc'.b)*(c)^2

T = (- 0.4*√fc'.b.H + As.Es.εcu) * (c) - As.Es.εcu.d

A =	(α.fc' b-0.5*b(α.fc')^2 /(Ec* εcu) + 0.4*b*√fc')	13859.63694
B =	(- 0.4*√fc'.b.H + As.Es.εcu)	345189.4335
D =	- As.Es.εcu.d	-217125352.1

c 1 =	-138.234996	Neglect	
c 2 =	113.3289011	OK	
a =	73.66378569	mm	
y° =	a / 2	rectangulare shape	
y° =	36.83189284		
Z =	d - y°		
Z =	309.1681072	mm	< 311.4
Cycle 2, Recycle with z	309.1681072	mm	
As =	902.9429639	mm^2	

A =	(α.fc' -0.5*(α.fc')^2 /(Ec* εcu) + 0.4*√fc')	13859.63694
B =	(- 0.4*√fc'.b.H + As.Es.εcu)	349719.5888
D =	- As.Es.εcu.d	-218692785.8

c 1 =	-138.8634143	Neglect	
c 2 =	113.6304597	OK	
a =	73.85979883		
y° =	a / 2		
y° =	36.92989941		
Z =	d - y°		
Z =	309.0701006	mm	< 309.1681072
Cycle 3, Recycle with z	309.0701006	mm	
As =	903.2292884	mm^2	

A =	(α.fc' -0.5*(α.fc')^2 /(Ec* εcu) + 0.4*√fc')	13859.63694
B =	(- 0.4*√fc'.b.H + As.Es.εcu)	349920.0159

D =	- As.Es.εcu.d			-218762133.6			
c 1 =	-138.8911826	Neglect					
c 2 =	113.6437669	OK					
a =	73.86844848	mm					
y° =	a / 2						
y° =	36.93422424	mm					
Z =	d - y°						
Z =	309.0657758	mm	Its equal to what assumed at first			309.07	OK
As(min) 1 =	(1.4/fy) * (Ac of depth = d)			173			
As(min) 2 =	0.25* √fc' / fy * (Ac of depth = d)			358.9425637			
As =	903.2292884	>	As(min) 1 & As(min) 2		OK	Use As =	904.2
c =	113.6437669						mm^2
εt =	0.006133805						
Ø = 0.9 as assumed we have to check it							
Ø	0.75 + 0.15 * (εt - εy) / (0.005 - εy)						
Ø	0.956690247						
check Ø Mn	Ø As * fy * (d - y°)						
Ø Mn	112.2887006	>	Mu	105.5229211	Ok		

Irregular shape beam Method			UHPCTB 9				
Ec =	50000	MPa					
H t =	405	mm					
H1 =	180	mm	Cover =	20	mm		
H 2 =	225	mm	bar =	25	mm		
d =	364.5	mm	bar 2 =	8	mm		
b =	150	mm					
L t =	1.9	m					
Overhange length (m)	0.16	m					
Ln =	1.58	m					
Load gap =	0.5	m					
shear span (a) =	0.54	m					
fc' =	135	Mpa					
fy =	420	Mpa					
Maximum L.L	445	kN				Two point loads	
wD =	1.19942	kN/m	Theoritecal FLAXURAL Failure's load			445	kN
PD =	0.75913		Experemental SHEAR failure's load			486	kN
Mu =	120.355	kN	Highest shear failure load			590	kN
Assume Z =	328.05	mm				1.32584	
Ø =	0.9	Factor					
As =	970.582	mm²					
α =	0.65						
εcu =	0.0035						
Es =	200000	MPa					
β =	0.65						
Find the value of (c) from C = T							
C = (α.fc'.b -0.5*(α.fc')^2 .b/(Ec* εcu) + 0.4*√fc'.b)*(c)^2							
T = (- 0.4*√fc'.b.H + As.Es.εcu) * (c) - As.Es.εcu.d							
A =	(α.fc' b-0.5*b(α.fc')^2 /(Ec* εcu) + 0.4*b*√fc')			13859.63694			
B =	(- 0.4*√fc'.b.H + As.Es.εcu)			397066.8125			
D =	- As.Es.εcu.d			-247643960			
c 1 =	-148.76	Neglect					
c 2 =	120.112	OK					
a =	78.0728	mm					
y° =	a / 2	rectangulare shape					
y° =	39.0364						
Z =	d - y°						
Z =	325.464	mm	<	328.05			
Cycle 2, Recycle with z	325.464	mm					
As =	978.295	mm²					
A =	(α.fc' -0.5*(α.fc')^2 /(Ec* εcu) + 0.4*√fc')			13859.63694			
B =	(- 0.4*√fc'.b.H + As.Es.εcu)			402465.9347			
D =	- As.Es.εcu.d			-249611940			
c 1 =	-149.5	Neglect					
c 2 =	120.465	OK					
a =	78.3023						
y° =	a / 2						
y° =	39.1512						
Z =	d - y°						
Z =	325.349	mm	<	325.4636019			
Cycle 3, Recycle with z	325.349	mm					
As =	978.64	mm²					
A =	(α.fc' -0.5*(α.fc')^2 /(Ec* εcu) + 0.4*√fc')			13859.63694			
B =	(- 0.4*√fc'.b.H + As.Es.εcu)			402707.4851			
D =	- As.Es.εcu.d			-249699985			
c 1 =	-149.54	Neglect					
c 2 =	120.481	OK					
a =	78.3126	mm					

y° =	a / 2						
y° =	39.1563	mm					
Z =	d - y°						
Z =	325.344	mm	Its almost equal to			325.349	OK
As(min) 1 =	(1.4/fy) * (Ac of depth = d)			182.25			
As(min) 2 =	(0.25* √fc' / fy) * (Ac of depth = d)			378.1345794			
As =	978.64	>	As(min) 1 & As(min) 2		OK	Use As=	981.7
c =	120.481						mm²
εt =	0.00608						
Ø = 0.9 as assumed we have to check it							
Ø	0.75 + 0.15 * (εt - εy) / (0.005 - εy)						
Ø	0.95381						
check Ø Mn	Ø As * fy * (d - y°)						
Ø Mn	127.947	>	Mu	120.3549647	Ok		

Irregular shape beam Method			UHPCTB 10
Ec =	50000	MPa	
H t =	405	mm	
H1 =	180	mm	
H 2 =	225	mm	
d =	364.5	mm	
b =	150	mm	
L t =	1.9	m	
Overhange length (m)	0.16	m	
Ln =	1.58	m	
Load gap =	0.3	m	
shear span (a) =	0.64	m	
fc' =	135	Mpa	
fy =	420	Mpa	
Maximum L.L	370	kN	
wD =	1.199423077	kN/m	
PD =	0.75912853		
Mu =	118.6429211	kN	
Assume Z =	328.05	mm	
Ø =	0.9	Factor	
As =	956.7753748	mm^2	
α =	0.65		
εcu =	0.0035		
Es =	200000	MPa	
β =	0.65		

Cover =	20	mm	
bar =	25	mm	
bar 2 =	8	mm	To forming of steel reinforcement cage

	Two point loads	
Theoritecal FLAXURAL Failure's load	370	kN
Experemental SHEAR failure's load	432	kN
Highest shear failure load	590	kN
	1.5945946	

Find the value of (c) from C = T

C = (α.fc'.b -0.5*(α.fc')^2 .b/(Ec* εcu) + 0.4*√fc'.b)*(c)^2

T = (- 0.4*√fc'.b.H + As.Es.εcu) * (c) - As.Es.εcu.d

A =	(α.fc' b-0.5*b(α.fc')^2 /(Ec* εcu) + 0.4*b*√fc')	10597.32014
B =	(- 0.4*√fc'.b.H + As.Es.εcu)	387402.2765
D =	- As.Es.εcu.d	-244121236.9

c 1 =	-171.1515485	Neglect		
c 2 =	134.5949218	OK		
a =	87.48669919	mm		
y° =	a / 2	rectangulare shape		
y° =	43.7433496			
Z =	d - y°			
Z =	320.7566504	mm	<	328.05
Cycle 2, Recycle with z	320.7566504	mm		
As =	978.5304882	mm^2		

A =	(α.fc' b-0.5*b(α.fc')^2 /(Ec* εcu) + 0.4*b*√fc')	10597.32014
B =	(- 0.4*√fc'.b.H + As.Es.εcu)	402630.8558
D =	- As.Es.εcu.d	-249672054.1

c 1 =	-173.6603398	Neglect		
c 2 =	135.6666914	OK		
a =	88.18334938			
y° =	a / 2			
y° =	44.09167469			
Z =	d - y°			
Z =	320.4083253	mm	<	320.7566504
Cycle 3, Recycle with z	320.4083253	mm		
As =	979.5942768	mm^2		

A =	(α.fc' b-0.5*b(α.fc')^2 /(Ec* εcu) + 0.4*b*√fc')	10597.32014
B =	(- 0.4*√fc'.b.H + As.Es.εcu)	403375.5078

D =	- As.Es.εcu.d			-249943479.7			
c 1 =	-173.7825699	Neglect					
c 2 =	135.7186535	OK					
a =	88.21712478	mm					
y° =	a / 2						
y° =	44.10856239	mm					
Z =	d - y°						
Z =	320.3914376	mm	Its equal to what we assumed at first			320.40833	OK
As(min) 1 =	(1.4/fy) * (Ac of depth = d)			182.25			
As(min) 2 =	(0.25* √fc' / fy) * (Ac of depth = d)			378.1345794			
As =	979.5942768	>	As(min) 1 & As(min) 2		OK	Use As =	981.7
c =	135.7186535						mm^2
εt =	0.005057109						
Ø = 0.9 as assumed we have to check it							
Ø	0.75 + 0.15 * (εt - εy) / (0.005 - εy)						
Ø	0.902855456						
check Ø Mn	Ø As * fy * (d - y°)						
Ø Mn	119.2688987	>	Mu	118.6429211	Ok		

Irregular shape beam Method			*UHPCTB 17+18*				
Ec =	*50000*	*MPa*					
H t =	*405*	*mm*					
H1 =	*180*	*mm*	*Cover =*	*20*	*mm*		
H 2 =	*225*	*mm*	*bar =*	*25*	*mm*		
d =	*364.5*	*mm*	*bar 2 =*	*8*	*mm*		
b =	*150*	*mm*					
L t =	*1.9*	*m*					
Overhange length (m)	*0.16*	*m*					
Ln =	*1.58*	*m*					
Load gap =	*0.3*	*m*					
shear span (a) =	*0.64*	*m*					
fc' =	*135*	*Mpa*					
fy =	*420*	*Mpa*					
Maximum L.L	*370*	*kN*	*Theoritecal Flexueal Failure's load*			*Two point loads*	
wD =	*1.199423077*	*kN/m*				*370*	*kN*
PD =	*0.75912853*		*Experemental SHEAR failure's load*			*565*	*kN*
Mu =	*118.6429211*	*kN*	*Highest shear failure load*			*590*	*kN*
Assume Z =	*328.05*	*mm*				*1.5945946*	
Ø =	*0.9*	*Factor*					
As =	*956.7753748*	*mm²*					
α =	*0.65*						
εcu =	*0.0035*						
Es =	*200000*	*MPa*					
β =	*0.65*						
Find the value of (c) from C = T							
C = (α.fc'.b -0.5(α.fc')^2 .b/(Ec* εcu) + 0.4*√fc'.b)*(c)^2*							
T = (- 0.4√fc'.b.H + As.Es.εcu) * (c) - As.Es.εcu.d*							
A =	*(α.fc' b-0.5*b(α.fc')^2 /(Ec* εcu) + 0.4*b*√fc')*			*10559.6102*			
B =	*(- 0.4*√fc'.b.H + As.Es.εcu)*			*387402.276*			
D =	*- As.Es.εcu.d*			*-244121237*			
c 1 =	*-171.4934554*	*Neglect*					
c 2 =	*134.8062796*	*OK*					
a =	*87.62408175*	*mm*					
y° =	*a / 2*	*rectangulare shape*					
y° =	*43.81204087*						
Z =	*d - y°*						
Z =	*320.6879591*	*mm*	<	*328.05*			
Cycle 2, Recycle with z	*320.6879591*	*mm*					
As =	*978.7400892*	*mm²*					
A =	*(α.fc' b-0.5*b(α.fc')^2 /(Ec* εcu) + 0.4*b*√fc')*			*10559.6102*			
B =	*(- 0.4*√fc'.b.H + As.Es.εcu)*			*402777.576*			
D =	*- As.Es.εcu.d*			*-249725534*			
c 1 =	*-174.0324154*	*Neglect*					
c 2 =	*135.8891915*	*OK*					
a =	*88.32797448*						
y° =	*a / 2*						
y° =	*44.16398724*						
Z =	*d - y°*						
Z =	*320.3360128*	*mm*	<	*320.69*			

Cycle 3, Recycle with z	320.3360128	mm					
As =	979.8154101	mm^2					
A =	(α.fc' b-0.5*b(α.fc')^2 /(Ec* εcu) + 0.4*b*√fc')			10559.6102			
B =	(- 0.4*√fc'.b.H + As.Es.εcu)			403530.301			
D =	- As.Es.εcu.d			-249999902			
c 1 =	-174.1562593	Neglect					
c 2 =	135.941752	OK					
a =	88.36213882	mm					
y° =	a / 2						
y° =	44.18106941	mm					
Z =	d - y°						
Z =	320.3189306	mm	Its almost equal to		320.336		OK
As(min) 1 =	(1.4/fy) * (Ac of depth = d)			182.25			
As(min) 2 =	(0.25* √fc' / fy) * (Ac of depth = d)			378.13			
As =	979.8154101	>	As(min) 1 & As(min) 2		OK	Use As =	982
c =	135.941752						mm^2
εt =	0.005043886						
Ø = 0.9 as assumed we have to check it							
Ø	0.75 + 0.15 * (εt - εy) / (0.005 - εy)						
Ø	0.902194316						
check Ø Mn	Ø As * fy * (d - y°)						
Ø Mn	119.1545893	>	Mu	118.64	Ok		

Irregular shape beam Method			*UHPCTB 19*				
Ec =	*50000*	*MPa*					
H t =	*405*	*mm*					
H1 =	*180*	*mm*	*Cover =*	*20*	*mm*		
H 2 =	*225*	*mm*	*bar =*	*25*	*mm*		
d =	*364.5*	*mm*	*bar 2 =*	*8*	*mm*		
b =	*150*	*mm*					
L t =	*1.9*	*m*					
Overhange length (m)	*0.16*	*m*					
Ln =	*1.58*	*m*					
Load gap =	*0.3*	*m*					
shear span (a) =	*0.64*	*m*					
fc' =	*135*	*Mpa*					
fy =	*420*	*Mpa*					
Maximum L.L	*378*	*kN*				*Two point loads*	
wD =	*1.19942*	*kN/m*	*Theoritecal FLAXURAL Failure's load*			*378*	*kN*
PD =	*0.75913*		*Experemental SHEAR failure's load*			*111*	*kN*
Mu =	*121.203*	*kN*	*Highest shear failure load*			*590*	*kN*
Assume Z =	*328.05*	*mm*				*1.5608*	
Ø =	*0.9*	*Factor*					
As =	*977.42*	*mm²*					
α =	*0.65*						
εcu =	*0.0035*						
Es =	*200000*	*MPa*					
β =	*0.65*						
Find the value of (c) from C = T							
C = (α.fc'.b -0.5(α.fc')^2 .b/(Ec* εcu)*							
*T = (As.Es.εcu) * (c) - As.Es.εcu.d*							
A =	*(α.fc' b-0.5*b(α.fc')^2 /(Ec* εcu))*			*13162.4999*			
B =	*As.Es.εcu*			*684194.037*			
D =	*- As.Es.εcu.d*			*-249388727*			
c 1 =	*-166.07*	*Neglect*					
c 2 =	*114.09*	*OK*					
a =	*74.1583*	*mm*					
y° =	*a / 2*	*rectangulare shape*					
y° =	*37.0792*						
Z =	*d - y°*						
Z =	*327.421*	*mm*	*<*	*328.05*			
Cycle 2, Recycle with z	*327.421*	*mm*					
As =	*979.298*	*mm²*					
A =	*(α.fc' -0.5*(α.fc')^2 /(Ec* εcu))*			*13162.4999*			
B =	*(As.Es.εcu)*			*685508.76*			
D =	*- As.Es.εcu.d*			*-249867943*			
c 1 =	*-166.26*	*Neglect*					
c 2 =	*114.179*	*OK*					
a =	*74.2163*						
y° =	*a / 2*						
y° =	*37.1082*						
Z =	*d - y°*						
Z =	*327.392*	*mm*	*<*	*327.42084*			
Cycle 3, Recycle with z	*327.392*	*mm*					
As =	*979.385*	*mm²*					
A =	*(α.fc' -0.5*(α.fc')^2 /(Ec* εcu))*			*13162.4999*			
B =	*(As.Es.εcu)*			*685569.472*			

D =	- As.Es.εcu.d			-249890073			
c 1 =	-166.27	Neglect					
c 2 =	114.183	OK					
a =	74.219	mm					
y° =	a / 2						
y° =	37.1095	mm					
Z =	d - y°						
Z =	327.391	mm	Its almost equal to			327.39	OK
As(min) 1 =	(1.4/fy) * (Ac of depth = d)				182.25		
As(min) 2 =	(0.25* √fc' / fy) * (Ac of depth = d)				378.1345794		
As =	979.385	>	As(min) 1 & As(min) 2		OK	Use As =	981.7
c =	114.183						mm²
εt =	0.00658						
Ø = 0.9 as assumed we have to check it							
Ø	0.75 + 0.15 * (εt - εy) / (0.005 - εy)						
Ø	0.97884						
check Ø Mn	Ø As * fy * (d - y°)						
Ø Mn	132.131	>	Mu	121.202921	Ok		

APPENDIX B

Antislip Aggregate

Non slip flooring aggregate

Description

Non-slip, chemically inert, graded, hard wearing aggregate available in four grades to suit most site requirements, they are identified as follows:

Antislip Aggregate No. 1 coarse
For use with Strongcoat SL, Strongcoat HB range and Gripdeck systems or any other coating systems to produce a coarse textured, non-slip floor topping.

Antislip Aggregate No. 2 medium
For use with Strongcoat HB, Strongcoat SL range and Gripdeck systems or any other coating systems to produce a medium coarse textured floor finish.

Anti

Applications

Antislip Aggregateare designed for use with Strongcoat resin products to produce non-slip industrial floors. Ideally suited for wet work areas in abattoirs, breweries, dairies, chemical industries, food processing areas, loading bays, ramps and walkways.

Advantages

- Range of products to suit most applications.
- Special grading to suit Strongcoat range products.
- Pre-packed ready for immediate site use.

Method of Use

Application Instructions

- All Antislip Aggregateshould be clean and dry prior to application.
- Antislip AggregateNo. 1, No. 2 and No. 3 are designed for use with Strongcoat HB, solvent free, resin based, roller applied floor coating, and Strongcoat TC2, and Strongcoat SL self-smoothing, solvent free epoxy floor toppings.
- In the case of application onto the Strongcoat SL products, a final coat of Strongcoat HB is applied.
- Antislip Aggregate No. 4 is used in conjunction with Strongcoat WD and Strongcoat EC10 floor coatings.

Application

The specially graded aggregates are “scattered” onto the first rolled coat of Strongcoat resin flooring whilst it is still wet. Sufficient Antislip Aggregate should be applied to completely cover or “blind” the surface.

The selected anti-slip grain should be allowed to fall vertically onto the resin coating rather than be thrown across the surface as this may cause bridges or scour the coatings, and damage the continuous film of the resin flooring.

When the first coat has dried, the excess aggregate can be brushed or vacuumed off the substrate and provided it is still clean and dry can be re-used.

The final roller coat of Strongcoat EC can then be applied to produce a hard wearing, chemically resistant non-slip floor. The texture and thickness of the floor is determined by the choice of the anti-slip grain.

Anti-slip grain	Finished floor	Finished floor thickness for Strongcoat HB
No. 1	Coarse	2.0 – 2.5 mm
No. 2	Medium	1.0 – 2.0 mm
No. 3	Fine	0.75 – 1.5 mm
No. 4	Extra Fine	0.3 – 0.6 mm

Packaging

Antislip aggregate is available in 25 kg bags.

Storage

Antislip aggregate has a shelf life of 12 months from date of manufacture if stored in dry conditions in the original unopened bags

If these conditions are exceeded, DCP Technical Department should be contacted for advise.

Antislip Aggregate

Cautions

Health and Safety

Antislip aggregate is non hazardous

Fire

Antislip aggregate is nonflammable.

More from Don Construction Products

A wide range of construc tion chemical products are manufactured by DCP which include:

- Concrete admixtures.
- Surface treatments
- Grouts and anchors.
- Concrete repair.
- Flooring systems.
- Protective coatings.
- Sealants.
- Waterproofing.
- Adhesives.
- Tile adhesives and grouts.
- Building products.
- Structural strengthening.

Hyperplast PC260

High performance concrete superplasticiser (Formerly known as Flocrete PC260)

Description

Hyperplast PC260 is a high performance super plasticising

Applications

- High strength and high performance concrete.
- Structures with congested reinforcement.
- Pre-cast concrete.
- Improved cohesion allow for use in mass concrete pours and piling.
- Self compacting concrete.

Advantages

- Optimizes cement utilization.
- High density and impermeable concrete through very high water reduction.
- Improves shrinkage and creep behaviors.
- Minimises segregation and bleeding problems by improving cohesion.
- Higher early and ultimate compressive strengths.
- Increases durability and resistance to aggressive atmospheric conditions thorough reduced permeability.

Compatibility

Hyperplast PC260 can be used with all types of Portland cement and cement replacement materials.

Hyperplast PC260 should not be used in conjunction with other admixtures unless DCP Technical Department approval is obtained.

Standards

Hyperplast PC260 complies with ASTM C494, Type A and G, depending on dosage used.

Method of Use

Hyperplast PC260 should be added to the concrete with the mixing water to achieve optimum performance.

Colour:Yellowish to brownish liquid	
Freezing point:≈ -7°C	
Specific gravity:1.1 ± 0.02	
Air entrainment:	Typically less than 2% additional air is entrained above control mix at normal dosages

An automatic dispenser should be used to dispense the correct quantity of Hyperplast PC260 to the concrete mix.

Dosage

The guidance dosage of Hyperplast PC260 is 0.5 - 3.0 litre per 100 kg of cementitious materials in the mix, including GGBFS, PFA or microsilica.

Representative trials should be conducted to determine the optimum dosage of Hyperplast PC260 to meet the performance requirements by using the materials and conditions in actual use.

Effects of Over Dosage

Over dosing of Hyperplast PC260 will cause the following:

- Significant increase in retardation.
- Increase in workability.

Ultimate concrete strength will not be adversely affected and will generally be increased provided that proper concrete curing is maintained.

Cleaning

Hyperplast PC260 can be washed with fresh cold water.

Packaging

Hyperplast PC260 is available in 25 litre pails, 210 litre drums and 1000 litre bulks supply.

Hyperplast PC260

Storage

Hyperplast PC260 has a shelf life of 12 months from date of manufacture if stored at temperatures between 2°C and 50°C.

If these conditi ons are exceeded, DCP Technical Department should be contacted for advice.

Cautions

Health and Safety

Hyperplast PC260 is not classified as hazardous material.
Hyperplast PC260 should not come into contact with skin and eyes.
In case of contact with eyes wash immediately with plenty of water and seek medical advice promptly.

For further information refer to the Material Safety Data Sheet.

Fire

Hyperplast PC260 is nonflammable.

More from Don Construction Products

A wide range of constructi on chemical products are manufactured by DCP which include:

- Concrete admixtures.
- Surface treatments
- Grouts and anchors.
- Concrete repair.
- Flooring systems.
- Protective coatings.
- Sealants.
- Waterproofing.
- Adhesives.
- Tile adhesives and grouts.
- Building products.
- Structural strengthening.

01-0023-A-2016

BUILDING TRUST

PRODUCT DATA SHEET
Sikadur®-330

2-COMPONENT EPOXY IMPREGNATION RESIN

PRODUCT DESCRIPTION

Sikadur®-330 is a 2-component, thixotropic epoxy based impregnating resin and adhesive.

USES

Sikadur®-330 may only be used by experienced professionals.
Sikadur®-330 is used as:
- Impregnation resin for SikaWrap® fabric reinforcement for the dry application method
- Primer resin for the wet application system
- Structural adhesive for bonding Sika® CarboDur® plates into slits

CHARACTERISTICS / ADVANTAGES

- Easy mix and application by trowel and impregnation roller
- Manufactured for manual saturation methods
- Excellent application behaviour to vertical and overhead surfaces
- Good adhesion to many substrates
- High mechanical properties
- No separate primer required

APPROVALS / STANDARDS

- Adhesive for structural bonding tested according to EN 1504-4, provided with the CE-mark

PRODUCT INFORMATION

Chemical Base	Epoxy resin	
Packaging	5 kg (A+B)	Pre-batched unit
Colour	Component A: white paste Component B: grey paste Components A + B mixed: light grey paste	
Shelf Life	24 months from date of production	
Storage Conditions	Store in original, unopened, sealed and undamaged packaging in dry conditions at temperatures between +5 °C and +30 °C. Protect from direct sunlight.	
Density	1.30 ± 0.1 kg/l (component A+B mixed) (at +23 °C)	
Viscosity	Shear rate: 50 /s	
	Temperature	**Viscosity**
	+10 °C	~10 000 mPas
	+23 °C	~6 000 mPas
	+35 °C	~5 000 mPas

TECHNICAL INFORMATION

Flexural E-Modulus	~ 3 800 N/mm² (7 days at +23 °C)	(DIN EN 1465)
Tensile Strength	~ 30 N/mm² (7 days at +23°C)	(ISO 527)
Tensile Modulus of Elasticity	~ 4 500 N/mm² (7 days at +23 °C)	(ISO 527)
Elongation at Break	0.9 % (7 days at +23 °C)	(ISO 527)
Tensile Adhesion Strength	Concrete fracture (> 4 N/mm²) on sandblasted substrate	(EN ISO 4624)
Coefficient of Thermal Expansion	4.5×10^{-5} 1/K (Temperature range −10 °C – +40 °C)	(EN 1770)

Glass Transition Temperature (EN 12614)

Curing time	Curing temperature	TG
30 days	+30 °C	+58 °C

Heat Deflection Temperature (ASTM D 648)

Curing time	Curing temperature	HDT
7 days	+10 °C	+36 °C
7 days	+23 °C	+47 °C
7 days	+35 °C	+53 °C

Resistant to continuous exposure up to +45 °C.

Service Temperature –40 °C to +45 °C

SYSTEM INFORMATION

System Structure

Substrate primer - Sikadur®-330.
Impregnating / laminating resin - Sikadur®-330.
Structural strengthening fabric - SikaWrap® type to suit requirements.

APPLICATION INFORMATION

Mixing Ratio	Component A : component B = 4 : 1 by weight When using bulk material the exact mixing ratio must be safeguarded by accurately weighing and dosing each component.
Consumption	See the "Method Statement for SikaWrap® manual dry application" Ref 850 41 02. Guide: 0.7 - 1.5 kg/m²
Ambient Air Temperature	+10 °C min. / +35 °C max.
Dew Point	Beware of condensation. Substrate temperature during application must be at least 3 °C above dew point.
Substrate Temperature	+10 °C min. / +35 °C max.
Substrate Moisture Content	< 4 % pbw

Pot Life (EN ISO 9514)

Temperature	Pot life	Open time
+10 °C	~90 minutes (5 kg)	~90 minutes
+23 °C	~60 minutes (5 kg)	~60 minutes
+35 °C	~30 minutes (5 kg)	~30 minutes

The pot life begins when the resin and hardener are mixed. It is shorter at high temperatures and longer at low temperatures. The greater the quantity mixed, the shorter the pot life. To obtain longer workability at high temperatures, the mixed adhesive may be divided into portions. Another method is to chill components A+B before mixing them (not below +5 °C).

APPLICATION INSTRUCTIONS

SUBSTRATE QUALITY

Substrate must be sound and of sufficient tensile strength to provide a minimum pull off strength of 1.0 N/mm^2 or as per the requirements of the design specification.
See also the "Method Statement for SikaWrap® manual dry application" Ref 850 41 02.

SUBSTRATE PREPARATION

Also refer to SikaWrap® Technical Information Manual for dry application method" Ref 850 41 02.

MIXING

Pre-batched units:
Mix components A+B together for at least 3 minutes with a mixing spindle attached to a slow speed electric drill (max. 300 rpm) until the material becomes smooth in consistency and a uniform grey colour. Avoid aeration while mixing. Then, pour the whole mix into a clean container and stir again for approx. 1 more minute at low speed to keep air entrapment at a minimum. Mix only that quantity which can be used within its pot life.

Bulk packing, not pre-batched:
First, stir each component thoroughly. Add the components in the correct proportions into a suitable mixing pail and stir correctly using an electric low speed mixer as above for
pre-batched units.

APPLICATION METHOD / TOOLS

Also refer to SikaWrap® Technical Information Manual for dry application method" Ref 850 41 02.

CLEANING OF TOOLS

Clean all equipment immediately with Sika® Thinner C. Cured material can only be removed mechanically.

LIMITATIONS

Sikadur®-330 must be protected from rain for at least 24 hours after application.
Ensure placement of fabric and laminating with roller takes place within open time.
At low temperatures and / or high relative humidity, a tacky residue (blush) may form on the surface of the cured Sikadur®-330 epoxy. If an additional layer of fabric or a coating is to be applied onto the cured epoxy, this residue must first be removed with warm, soapy water to ensure adequate bond. In any case, the surface must be wiped dry prior to application of the next layer or coating.
For application in cold or hot conditions, pre-condition material for 24 hours in temperature controlled storage facilities to improve mixing, application and pot life limits.
For further information on over coating, number of layers or creep, please consult a structural engineer for calculations and see also the "Method Statement for SikaWrap® manual dry application" Ref 850 41 02.
Sikadur® resins are formulated to have low creep under permanent loading. However due to the creep behaviour of all polymer materials under load, the long term structural design load must account for creep. Generally the long term structural design load must be lower than 20-25% of the failure load. Please consult a structural engineer for load calculations for the specific application.

VALUE BASE

All technical data stated in this Product Data Sheet are based on laboratory tests. Actual measured data may vary due to circumstances beyond our control.

LOCAL RESTRICTIONS

Please note that as a result of specific local regulations the performance of this product may vary from country to country. Please consult the local Product Data Sheet for the exact description of the application fields.

ECOLOGY, HEALTH AND SAFETY

For information and advice on the safe handling, storage and disposal of chemical products, users shall refer to the most recent Safety Data Sheet (SDS) containing physical, ecological, toxicological and other safety-related data.

LEGAL NOTES

The information, and, in particular, the recommendations relating to the application and end-use of Sika products, are given in good faith based on Sika's current knowledge and experience of the products when properly stored, handled and applied under normal conditions in accordance with Sika's recommendations. In practice, the differences in materials, substrates and actual site conditions are such that no warranty in respect of merchantability or of fitness for a particular purpose, nor any liability arising out of any legal relationship whatsoever, can be inferred either from this information, or from any written recommendations, or from any other advice offered. The user of the product must test the product's suitability for the intended application and purpose. Sika reserves the right to change the properties of its products. The proprietary rights of third parties must be observed. All orders are accepted subject to our current terms of sale and delivery. Users must always refer to the most recent issue of the local Product Data Sheet for the product concerned, copies of which will be supplied on request.

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ISO 14001 Environmental Management EMS 45308
OHSAS 18001 Occupational Health and Safety Management OHS 585274

الخلاصة

ينقسم العمل في هذه الرسالة الى جزأين، الأول يتعلق بالخرسانة فائقة الأداء UHPC ، و هي نوع من الخرسانة الذي تم تطويره لتعزيز مرونة و متانة الهياكل الخرسانية. إستخدام المواد المتوفرة محلياً يعتبر خطوة أساسية لتوفير المواد، الطاقة، و تقليل تكلفة الخرسانة. في الدراسة الحالية، تم دراسة تأثير كل من محتوى المادة الرابطة، نسبة الماء/السمنت، و تدرجات الرمل على مقاومة الإنضغاط للخرسانة. من الممكن تطوير خلائط UHPC من المواد المتوفرة محلياً و بإستخدام ثلاثة أنواع من الرمل (الرمل #2، الرمل #3، الرمل #4)، تم الحصول على مقاومة إنضغاط للمكعبات (163.2 MPa، 164.8MPa، 167 MPa) بإجمالي محتوى مادة رابطة (1250 كجم/م^3، 1300 كجم/م^3، و 1300 كجم/م^3) من الرمل #3 مع (25%، 30%، و 30%) على التوالي من دخان السليكا. و مقاومة الإنضغاط الإسطوانية (160 MPa، 150.9 MPa، 158.8MPa) بإجمالي محتوى مادة رابطة (1337.7 كجم/م^3، 1250 كجم/م^3، و 1337.7 كجم/م^3) من ثلاثة أنواع من الرمل (الرمل #2، الرمل #3، الرمل #4) على التوالي، مع (30%، 25%، و 30%) على التوالي من دخان السليكا. كما تبين من النتائج التي تم الحصول عليها كانت مقاومة الإنضغاط للإسطوانات الخرسانية أقل بـ 12% من مقاومة الإنضغاط للمكعبات الخرسانية. بعد الحصول على خليط UHPC تم إستخدامه في الجزء الثاني (الهدف الرئيسي) حيث تم إختبار تسعة عشر عتبة غير موشورية (إثنا عشر مجموعة، تم إختبارها تحت تأثير حملين مركزين) مع المتغيرات التالية قضبان البوليمرات المسلحة القريبة من السطح NS CFRP بثلاثة إتجاهات (0˚، 30˚، و 45˚)، شرائط CFRP تم أستخدامها على شكل حرف U بنفس إتجاهات الـ NS، عدد الركائب، زاوية ميل العتبات الغير موشورية، نسبة فضاء القص الى العمق الفعال a/d، نسبة حديد التسليح للشد، نسبة الألياف الفولاذية، و عدد الفتحات و موقعها. أظهرت النتائج أن قضبان و شرائح الـ CFRP المائلة أكثر كفاءة من الرأسية. كان الـ NS الأكثر فاعلية، حيث لم يقتصر الأمر على زيادة مقاومة القص للعتبات الغير موشورية بنسبة (11.3%، 35.4%، و 36.6%) بالإتجاهات (0˚، 30˚، و 45˚) على التوالي، و لكن أيضا زاد حمل التشقق الأول (3.3%، 43.7%، و 57.4%) على التوالي، الهطول الخدمي زاد بنسبة (210%، 222%، 225%) على التوالي، و الهطول الكلي زاد بمقدار (11.5%، 85.6%، و 99%) على التوالي. يعتبر الـ NS أكثر فاعلية من الركائب في زيادة مقاومة القص، و حمل التشقق الأول، و الإنحراف بنسبة (13.9%، 18%، و 53.4%) على التوالي، مقارنة مع نفس عدد القضبان، بالرغم من أنه كان قطر قضبان الركائب 8 ملم و قطر قضبان الـ NS 6 ملم. تعتبر قضبان الـ NS أكثر ملائمة من شريط الـ CFRP و بجميع الإتجاهات في زيادة سعة الحمل الأقصى، حمل التشقق الأول، الهطول الخدمي، و الهطول النهائي بنسبة (21.1%، 40.5%، 73.3%، و 93.8%) على التوالي للإتجاه 45˚. تتأثر مقاومة القص للعتبات الغير موشورية بعدد الركائب و زادت من 19.9% الى 30.8% عند إستخدام خمسة ركائب بدلاً من أربعة ركائب، و زاد حمل التشقق الأول، هطول الخدمي، و الهطول النهائي بنسبة (43.7%، 240%، و 56.5%) على التوالي، عند زيادة عدد الركائب من (0 الى 5). كان للميل تأثير إيجابي على مقاومة القص للعتبات غير الموشورية. تمت زيادة حمل الفشل، حمل التشقق الأول، و الهطول بنسبة (19.3%، 24.5%، و 86.5%) على التوالي، عند زيادة زاوية الميل من 9.7˚ الى 15.9˚. أدت زيادة a/d الى تقليل مقاومة القص للعتبات الغير موشورية. عندما قلت نسبة الـ a/d من 2.94 الى 2.3 أدت الى حمل الفشل و الهطول بمقدار 10.6% و 50.4% على التوالي. كان لحديد تسليح الشد تأثير إيجابي خاصة عندما يتم توزيع أو نشر القضبان بصفين بدلاً من صف واحد، عندما تغيرت مساحة الحديد من (981.7 ملم^2 الحديد منشور بصف واحد) الى

(804.2 ملم^2 الحديد موزع على صفين) بالرغم من أن مساحة الحديد قد قلت بمقدار 18% إلا أن مقاومت القص زادت بمقدار 3.2% كما زاد كل من حمل التشقق الأول، الهطول تحت تأثير الحمل الخدمي، و الهطول النهائي بنسبة (12%، 90.2%، و 6.6%) على التوالي. عند زيادت نسبة حديد التسليح للشد من 1.22% الى 1.57% (في كلا الحالتين قضبان الشد موزعة على صفين) زاد كل من الحمل النهائي، خمل التشقق الأول، الهطول تحت تأثير الحمل الخدمي، و الهطول النهائي بنسبة (20.5%، 26.5%، 27%، و 41%) على التوالي. تتمتع الألياف الفولاذية (بنسبة 2% من حجم الكلي للخليط) بفاعلية ممتازة في زيادة مقاومة القص للعتبات غير الموشورية بنسبة 300%، أي أنها أفضل من الركائب، أشرطة الـ CFRP ، و كذلك أفضل من قضبان الـ NS CFRP. أدى وجود الألياف الفولاذية الى زيادة كبيرة في قيمة الحمل الذي ظهر عنده التشقق الأول بنسبة 84.7%، و زيادة الهطول الكلي بمقدار 235%. كان لوجود الفتحات تأثير سلبي على مقاومة القص للعتبات غير الموشورية ليس فقط من ناحية سعة التحمل، و لكن أيضاً تؤثر على حمل التشقق الأول، الهطول الكلي، و كذلك الهطول تحت تأثير الحمل الخدمي بنسبة (5.2%، 18.2%، 13.5%، و 19%) على التوالي. تتمتع العتبة ذات الفتحة الواحدة في المنطقة الموشورية بنفس سعة القص و الهطول الكلي للعتبة ذات الفتحتين. تتراوح زاوية الفشل بين 31.197° الى 36.297° للعتبات غير الموشورية التي لا تحتوي على (ركائب، شرائط و قضبان الـ CFRP). أما بالنسبة للعتبات الغير موشورية التي تحتوي على (ركائب، شرائط و قضبان الـ CFRP) فقد تراوحت زاوية الفشل بين 41.197° الى 52.797°. أخيراً تم التصميم بإستخدام ثلاثة طرق (طريقة العتبة العميقة، طريقة الأشكال غير المنتظمة، و صيغ ناصر و صيغة البغمبرلي و آخرون لحساب مساهمة حديد التسليح الطولي في سعة القص)، و مقارنةً بالنتائج التجريبية لجميع العتبات غير الموشورية (التسعة عشر) كانت صيغ ناصر مناسبة لتصميم هذا النوع من العتبات بمعدل نسبة مطابقة 93.3%.

جمهورية العراق

وزارة التعليم العالي و البحث العلمي

جامعة ميسان / كلية الهندسة

سلوك القص للعتبات غير الموشورية ذات الفتحات الطولية بإستخدام الخرسانة فائقة الاداء

اطروحة

مقدمة الى قسم الهندسة المدنية في جامعة ميسان كجزء من متطلبات نيل شهادة الماجستير

في علوم الهندسة المدنية / إنشاءات

من قبل

حيدر جبار جحيل

(بكالوريوس هندسة مدنية 2005)

بإشراف

د. ناصر حكيم طعمة

2020