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Behavior of composite concrete-cellular steel beams under combined flexure and torsion

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ABSTRACT: The experimental evaluation of the behavior of nine composite concrete-cellular steel beams under combined effect of flexure and torsion was examined up to failure, where two strengthening technics were implemented: strengthening by intermediate stiffeners only or by both external prestressing and intermediate stiffeners. The experimental results show that adding intermediate stiffeners only improved the load carrying capacity by 21.8% for specimens under pure bending moment, 33.3% for specimens under combined effect of flexure and torsion, and 4.44% for specimens under pure torsion, respectively. The other strengthening technique enhanced the load carrying capacity by 134.3 %, 116.6% and 4.88%, respectively. It is worth to mention that, the first technique reduced the midspan deflection under service loads by 22% and 13% for specimens under pure bending and combined flexure and torsion, respectively. While the second technique decreased, the mentioned values by 61% and 44%, respectively.

1 INTRODUCTION

Castellation comes from the appearance of the beam and its similarities with castle battlements. Castellated beams can be manufactured from either hot-rolled or built-up I- H- or U-sections. Since the invention of the castellated beams, many attempts have been made to develop the shape of castellation. It can take different shapes such as hexagon, octagon, circular, square and sinusoidal shape, Afefy, et al (2012).

Castellated cellular beams are widely implemented in tall buildings to provide longer spans and to achieve passage for ducts, cables and other utilities reducing the height of the floor assembly. Cellular beams consist of steel sections with distributed circular web openings that are produced by two different methods. The first method, the Westok method, consists on twice cutting an original hot-rolled beams web in a half circular pattern along its web centerline. The split halves are shifted a half-pitch in relation to one another and then welded together at the tops of the teeth to form deeper beam with full circular shaped web openings. The second method, the Febsec method, consists on cutting circular openings in a steel plate and the cellular beams are fabricated from welding three different fabricated steel plates. The finished depth, opening spacing and opening diameter are flexible to achieve the design requirements. These openings up of the original rolled beam may lead to substantial increase the overall beam depth, section modulus, moment of inertia and bending rigidity, while reducing the overall weight of the beam, Durif et al (2013). A cellular beam can have a bending resistance up to 2.5 times higher than its parent solid section and so improve the cost efficiency of a design. The cutting process, according to Westok's method, and an application of cellular beams in buildings are illustrated in Figure (1) and Plate (1).

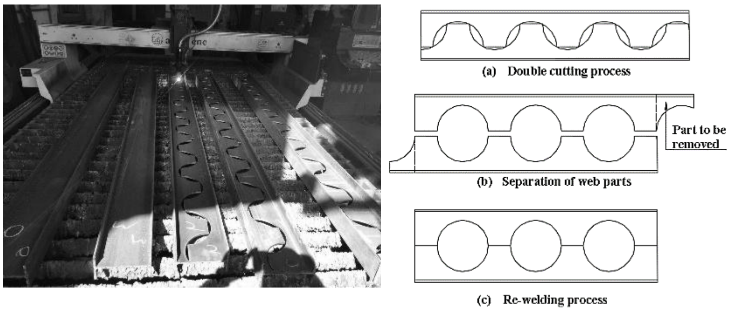


Figure 1. Manufacturing of cellular beams

Plate 1. Cellular beams in buildings

Cellular beams are a modern version of the traditional castellated beams. The emergence of cellular beams was firstly for architectural application, where exposed steelwork with circular web openings in the beam was consider aesthetically pleasing more than castellated beams. Cellular beams are also use as roof beams beyond the range of portal-frame construction, and are perfect solutions for curved roof applications, combining weight savings with a low-cost manufacturing process, Erdal and Saka (2013).

Composite concrete - cellular steel beams have been use in various designs of structural buildings. The existence of opening in the web alters the stress distribution within the member and influences its collapse behavior. Accordingly, the strength of these beams may be governed by the plastic deformation that occur due to moment and shear at openings.

The load carrying capacity of the composite concrete – cellular steel beams will be reduced at the opening because of the reduction in the contribution of web to the moment capacity. This is not very significant, as usually the contribution of the web to the moment capacity is very limited. However, the reduction in shear capacity at the opening can be significant. Therefore the ultimate capacity under the combined action of moment and shear at the section where there is an opening will be less compared to that at the normal cross section without opening.

To restore the strength lost, strengthening with intermediate steel stiffeners, which should be welded to the web between openings, could be used. However, compared to such scheme, the use of post-tensioned external tendons to strengthen these girders is relatively simple technique and economic method to employ also.

Few studies have been conducted on the behavior of composite concrete-cellular steel beams under combined flexure and torsion. In the current study, the circular castellation has been chosen to show the effect of different strengthening techniques on the global behavior of such configuration under the effect of flexure and torsion.

The objective of this investigation was to examine experimentally the benefits of steel stiffeners and external prestressing as used in strengthening of composite concrete – cellular steel section flexural members under combined flexure and torsion and to evaluate their effect on both the service load behavior and nominal carrying capacity.

2 EXPERIMENTAL PROGRAM AND SPECIMEN FABRICATION

To investigate the flexure – torsion interaction diagram and deformability, the experimental program was design in such way that nine composite concrete - cellular steel specimens were fabricated and tested as simply supported over an effective span of 2900 mm. All specimens had the same cross – sectional dimensions where the total height of the beam was 360 mm divided into 60 mm concrete deck slab and 300 mm hot-rolled cellular I-beam as the steel section.

The steel IPEA - section with overall height of 200 mm was chosen to fabricate the cellular steel section where the castellation pattern was drawn on the web to obtain circular castellation of 209.55 mm diameter. Electric cutting was then used to split the web into two parts where one part was shifted half a pitch to obtain the shape of castellation. Finally, the steel section was assembled using continuous electric welding of 3 mm thickness on both sides, the end of the beam was then polished and the extra parts of castellation were cut to obtain a cellular I-section of total height of 300 mm and total length of 3000 mm.

The success of composite action depends on the shear resistance at the interface between the steel beam and the cast-in-place deck to allow full transfer of stresses. Thus, a good connection between the two components of the composite system is essential. To achieve full connection between the steel cellular I-section and the concrete deck slab, rigid shear connectors were used in form of channel section of total height of 40 mm and flange width of 30 mm. The thickness of the channel was constant for both web and flange and was equal to 3 mm, while its spacing was 150 mm perpendicular to the longitudinal axis of the beam. The length of the channel was chosen as 50 mm centered above the upper flange. The concrete deck slab above the steel cellular I-beam was 60 mm in thickness and 500 mm in width. It was reinforced in both directions by mild steel rods of 6 mm diameter and spaced every 150 mm. Figure (2) shows details of the section after castellation.

For specimens subjected to external prestressing, two low - relaxation seven - wire steel strands, with 12.7 mm diameter and Grade (270) were used and located at symmetrical distances of 100 mm from either sides relative to the longitudinal axis of the composite beam. The two strands were tensioned simultaneously from one end. Special care was exercised to balance the prestressing force in the strands to avoid biaxial bending of the specimens. The wedge anchored prestressing strands were supported directly on an 200 mm wide x 300 mm deep x 20 mm thick bearing plate attached to the ends of the beam, where two holes were formed to facilitate the application of the prestressing. For beams exposed to pure bending moment or to combined effect of flexure and torsion, draped external tendon profile was adapted. While straight profile was used for specimens subjected to pure torsion (Fig. 2). A tapered bearing plate was used for the deviated strands to comply with their inclined profile. The two deviators around the third points of the span consisted of 25 mm diameter cylindrical rod welded to the bottom surface of the steel section. The steel strands were stretched and anchored to the vertical steel bearing plate using Supreme Product anchorage chucks (grips). The prestressing force was transferred to the steel strand through the use of hydraulic jack of capacity 20 tons and a stroke of 150 mm. The target effective prestress f_{pe} in the external prestressing steel was 40% of the ultimate strength of the strand f_{pu}. The corresponding stress was monitored accurately using readings of the strain gauges attached to the surface of the prestressing strands along with the readings of the pressure gauge of the hydraulic jack used in the prestressing operation. The longitudinal prestressing steel strands, welded wire fabric of slab reinforcement, and steel for web, flanges, stiffeners, and shear connectors had different mechanical properties which are summarized at Table 1.

The concrete mix was prepared using Type I cement, crushed stone of 10 mm maximum aggregate size and fine river sand. The aggregate: sand: cement proportions by weight were 3:1.5:1 with water – cement ratio equal to 0.45. The concrete compressive strength was determined from three standard 150 x 150 x 150 mm cubes taken from each specimen, where the target value was 32 MPa. The concrete deck for all specimens was cast horizontally in wooden forms. One day after casting, the standard cubes and the sides of the deck slab were

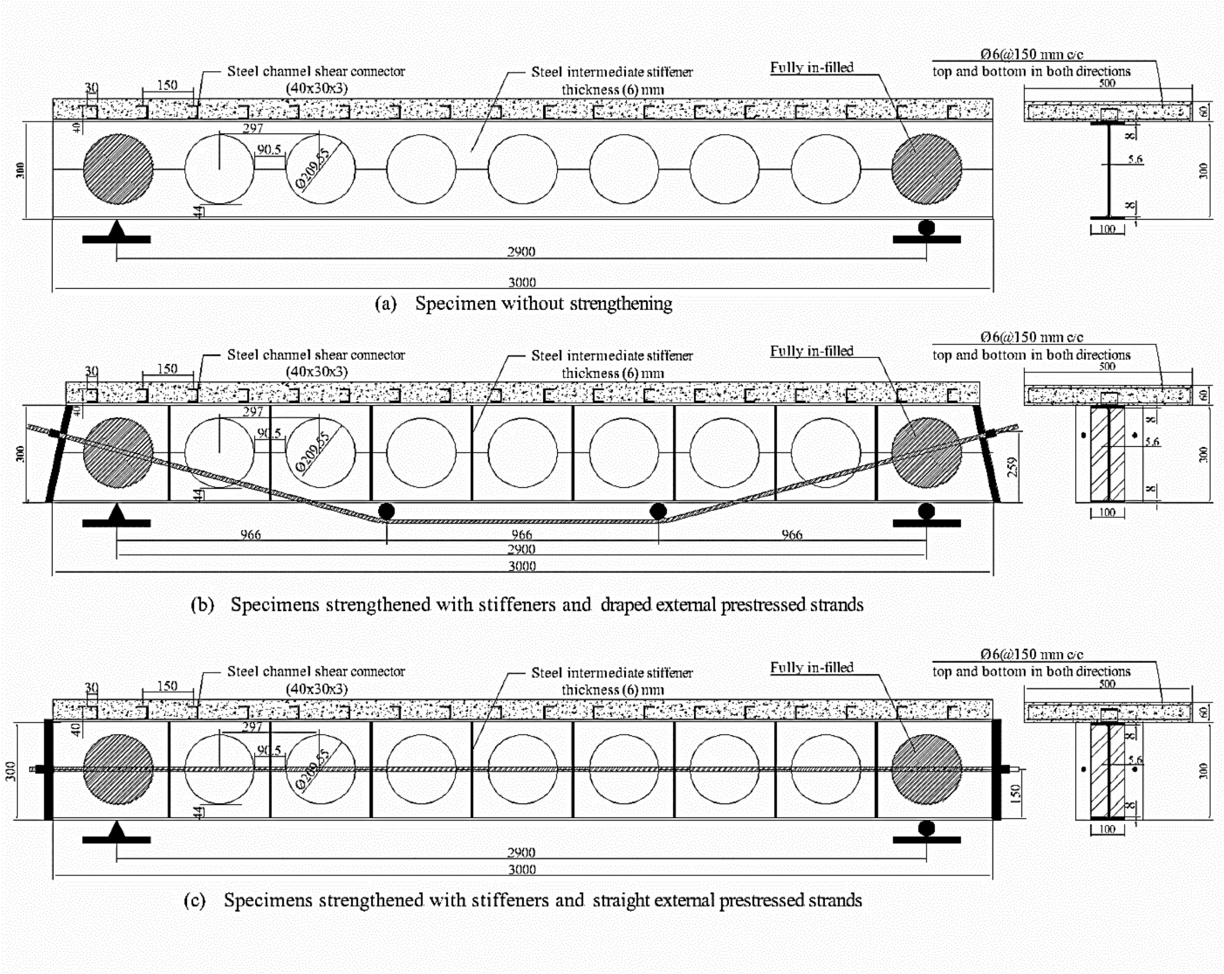


Figure 2. General details for specimens.

stripped from the molds and covered in wet material. The deck slabs were covered until the seventh day, when the wet material was removed and the specimen allowed air-drying until testing. It should be mentioned that the external prestressing was applied to the steel cellular I-section before pouring the concrete of the deck slab. The test specimens were divided into three groups according to applied loads (pure bending (PB), combined flexure and torsion (FT), and pure torsion (PT)). Each group has three specimens, which designated according to the type of the strengthening techniques used. The first specimen of each group, (CEBR), has no strengthening and considered as a control specimen. The second specimen of each group, (CEBS), was characterized by strengthening using vertical intermediate steel stiffeners at both sides of all web-posts. While the third specimens, (CEBP), were subjected to external prestressing in addition to using vertical intermediate steel stiffeners. It is worth to mention that, all specimens were strengthened at the end zone by filling the first castellation.

3 INSTRUMENTATION AND TESTING PROCEDURE

All specimens were extensively instrumented. Load cell monitored the total applied load. At each loading stage stresses in concrete, prestressed strands and steel cellular section at different locations and levels were monitored using electrical resistance strain gauges. These strain gauges were installed at different levels of the steel web, steel flange section and at the extreme

Table 1. Mechanical properties of steel components

Component	Thickness (mm)	Yield Stress (MPa)	Ultimate Stress (MPa)	Elongation (%)
Plate for web	5.6	397.87	520.52	26.60
Plate for top and bottom flanges	8.0	378.95	519,80	27,15
Plate for stiffeners	6.0	394.71	520,52	26,38
Plate for shear connectors	3.0	246.14	332,79	29,26
Welded wire fabric	Φ = 6.0	456.62	615,60	20,12
Prestressing steel strand	Φ = 12.7	1675.00	1862,00	6,00

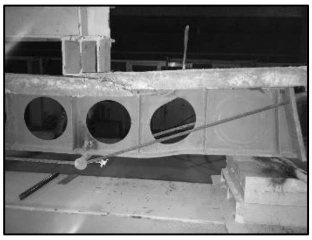
top and bottom concrete fibers of the deck slab. Additionally, for specimens exposed to pure bending moment and specimens under combined effect of bending moment and torsion, midspan deflections were measured manually with dial gauges. Meanwhile, for specimens under combined flexure and torsion and specimens subjected to pure torsion, simple mechanism was installed with two dial gauges to record the upward and downward displacement due to the applied load, which will be used to determine the twisting angles of the beam cross-section. The applied supports allowed angular movement in one end and both horizontal and angular movement of the beam at the other end, and, hence simulated simply supported scheme. All beams of the first and second groups were loaded in four-point bending using two symmetrical concentrated loads applied at one-third the span length. While for specimens of the third group the two concentrated loads were applied at the ends of the span to create pure torsion at these ends. The test was conducted using closed loop ram with 50 ton capacity actuator. Specimens were subjected to a monotonically increasing load to failure using a load control test. All measurements, such as beam deflections, twisting angles, strains in concrete, steel strands and cellular steel section were recorded twice, immediately after the application of the load and after 10 minutes later. The total testing time took an average of three hours, depending on the pattern of loading and the strengthening scheme.

4 EXPERMINTAL RESULTS AND DISCUSSIONS

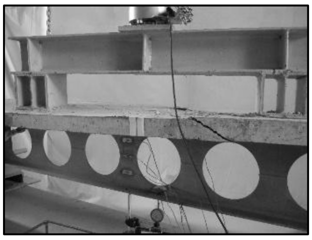
Detailed analysis and comparison among the three groups have been conducted considering the following: mode of failure; carrying capacity; load-deformation responses. Eight failure modes for castellated beams are known, Lorenc and Kubica (2006): flexural mechanism; lateral torsional buckling; distortional buckling; web-post buckling due to shear force; web-post buckling due to compression force; Vierendeel or shear mechanism; rupture of welded joints; and ultimate deflection. In the current investigation, the failure due to lateral torsional buckling was not happened due to the stiffness achieved by the concrete deck slab. As for the rupture of welded joints, this mode of failure was excluded as the welded joints were properly designed. Also, failure due to ultimate deflection could not have been possible to observe since the tested specimens were loaded until the complete collapse had occurred. The remaining modes of failure occurred individually or in combined modes. For composite concrete – cellular steel beams under puretorsion another mode of failuer can be observed due to the cantilever effect mechanism for the concrete deck slab. For the control specimen under pure bending CEBR-PB, the failure was characterized by local buckling of web-post, at section located directly under the concentrated load, due to shear force and Vierendeel mechanism. The remaining specimens of

(a) CEBR-PB

(b) CEBS-PB

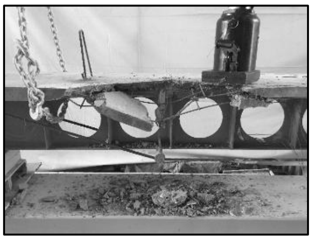


(c) CEBP-PB



(d) CEBR-FT

(e) CEBS-FT



(f) CEBP-FT

(g) CEBR-PT

(h) CEBS-PT

(i) CEBP-PT

Figure 3. Failure modes of experimental specimens.

group (I) failed by a combination of failure modes such as flexural mechanism and Vierendeel or shear mechanism. It is worth to mention that at failure of specimens of group (II) diagonal cracks were formed at the top surface of the deck slab and rotation occurred about an axis near the top surface. The angle of cracks were found to be close to 45^{o}. The widest crack on the top surface was inclined and extended across the depth of the deck slab. Thus a hinge was formed, the failure was well defined and followed by crushing of concrete (Fig. 3 d, e, f). In all specimens of group (III) which subjected to pure torsion, diagonal cracks also formed at the top surface with angle close to 35^{o} but before complete collapse a triangle shaped area of tensional stress in the concrete deck slab can be seen near supports (Fig. 3 g, h, i). This is because, as load increased, the concrete slab increasingly acts as a cantilever to resist the external load. The triangle shaped area typically describes the area of crack formation in a cantilever orientation. Since the concrete and its reinforcement was not designed for a cantilever orientation, the slab had a low ability to resist tension within this area, therefore, the adopted 175 mm eccentricity experienced failure in this region earlier due to the cantilever effect increasing, i.e. the moment around that axis increasing. So from this behavior, it can be assumed that the specimens of group (III) were behaving in an appropriate maner and that the problem exists in the estimation of the deck slab thickness. It was noted that, using these strengthening systems excluded certain failure modes, which are commonly happened with cellular beams. Table 2 summarizes the recorded collapse characteristics for all specimens after complete failure along with the dominant modes of failure.

Table 2. Experimental load carrying capacity of the tested specimens

Group	Specimen designation	P_u (kN)	M_u (kN.m)	M_u / $M_{u,ref}$	T_u (kN.m)	T_u / $T_{u,ref}$	Mode of failure
I	CEBR-PB	160.0	77.33	1.000			Web-post buckling due to shear force plus Vierendeel mechanism
	CEBS-PB	195.0	94,25	1,218			Flexural mechanism
	CEBP-PB	375.0	181,25	2,346			Vierendeel or shear mechanism
II	CEBR-FT	90.0	43,50	1,000	7.875	1.000	Combined flexural and shear mechanisms
	CEBS-FT	120.0	58,00	1,333	10.50	1,333	Combined flexural and shear mechanisms
	CEBP-FT	195.0	94,25	2,166	17.063	2,166	Combined flexural and shear mechanisms
III	CEBR-PT	45.0			3.950	1,000	Cantilever effect mechanism
	CEBS-PT	47.0			4.123	1,044	Cantilever effect mechanism
	CEBP-PT	47.2			4.130	1,048	Cantilever effect mechanism

It may be seen that, in comparison with reference beam, the first strengthening technique improved the load carrying capacity by 21.8% for specimens under pure bending moment, 33.3% for specimens under combined effect of flexure and torsion, and 4.44% for specimens under pure torsion, respectively. While the application of external prestressing along with the steel stiffeners resulted in increasing the ultimate load by 134.3% for specimens under pure bending moment, 116.6% for specimens under combined effect of flexure and torsion, and 4.88% for specimens under pure torsion, respectively. Figure 4 illustrates the flexure – torsion interaction diagram. The strain increment in the external prestressed strands due to applied loads in specimens CEBP-PB and CEBP-FT was shown in Fig. 5, where these increments attained at failure 1526 x 10^{-6} and 1100 x 10^{-6}, respectively.

It is should be mention that, first technique reduced the midspan deflections under service loads by 22% and 13% for specimens under pure bending moment or combined flexure and torsion, respectively. While the second technique decreased, the mentioned values by 61% and 44% for specimens under pure bending moment or combined flexure and torsion, respectively (Fig. 6).

In term of twisting angle at serviceability stage, it can be concluded that specimens strengthened by intermediate stiffeners and exposed to combined flexure and torsion or pure torsion only has little contribution in term of torsional resistance compared with reference specimens. Accordingly, for specimens under combined effect of flexure and torsion, the first technique reduced the twisting angle by 6.25%, while the second technique decreased, the mentioned value by 13.33%, respectively. Meantime for specimens under pure torsion, the strengthening technique by steel stiffeners reduced the twisting angle under service loads by 7.69% while the second technique with addition of external prestressing decreased the twisting angle by 16.67%, respectively (Fig. 7).

In summary, therefore, it may be said that external prestressing may enhance the failure characteristics; however, it reduces the ductility of the system leading to a matter of choice for deciding upon the suitability of the performance criteria of the structural system.

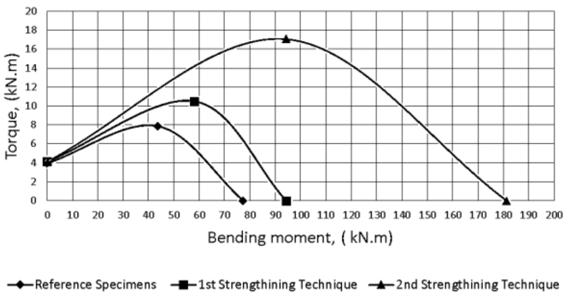


Figure 4. Flexure - torsion interaction diagram.

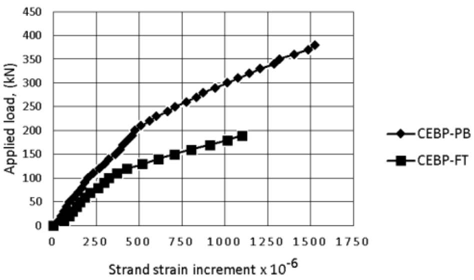


Figure 5. Load - strand strain increment diagram.

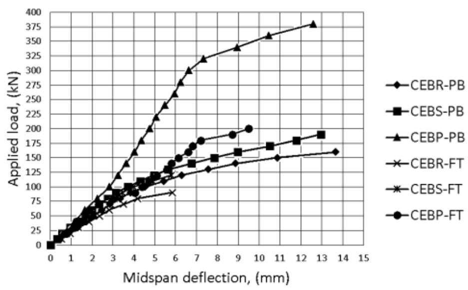


Figure 6. Load - midspan deflection of specimens.

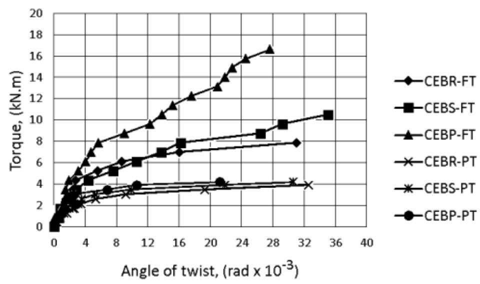


Figure 7. Torque - angle of twist of specimens.

5 CONCLUSIONS

Based on the studied configuration of strengthening the composite cellular beams subjected to incremental monotonic static loading, the following conclusions can be drawn:

1. Adding vertical stiffeners at all web posts leads to increasing the ultimate capacity by 21.8%, 33.3% and 4.44% for specimen under pure bending, specimen subjected to combined flexure and torsion and specimen exposed to pure torsion, respectively.
2. Using external prestressing in addition to vertical stiffeners leads to increase in the ultimate load capacity by 134.3%, 116.6% and 4.88% for specimen under pure bending, specimen with combined flexure and torsion and specimen exposed to pure torsion.
3. Strengthening with external prestressing noticeably leading to decrease midspan deflection and twisting angle when compared with reference group and group with vertical stiffeners.

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