Citation
Pham, T.M. and Chen, W. and Elchalakani, M. and Karrech, A. and Hao, H. 2020. Experimental investigation on lightweight rubberized concrete beams strengthened with BFRP sheets subjected to impact loads. Engineering Structures. 205: ARTN 110095. http://doi.org/10.1016/j.engstruct.2019.110095

Experimental Investigation on Lightweight Rubberized Concrete Beams Strengthened with BFRP Sheets Subjected to Impact Loads

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Abstract

This study experimentally investigates the impact behaviour of rubberized concrete beams strengthened with basalt fiber reinforced polymer (BFRP). Twelve reinforced concrete beams, which consisted of different rubber contents (0%, 15%, and 30%), were tested under impact loads. Various wrapping schemes were considered to determine the most effective strengthening schemes for impact resistance performance of both the conventional and rubberized concrete beams. The experimental results have shown that rubberized concrete had 10-18% higher imparted energy per unit weight than that of normal concrete. The rubberized concrete beams localized the damage at the impact area and slowed down the stress wave velocity. Although rubberized concrete beams possessed lower compressive strength (50.3 MPa, 25.4 MPa and 14.7 MPa for beams with 0%, 15% and 30% rubber content, respectively), they yielded less displacement as compared to the reference beams under the same impact velocity. The rubberized concrete beams experienced a lower peak impact force. Meanwhile, the use of U-shape BFRP wraps concentrating at the impact area

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showed similar performance as those with BFRP wraps uniformly distributed along the entire beam, therefore, this proposed strengthening scheme provides a cheaper solution for strengthening concrete structures.

Keywords: Rubberized concrete; Fiber Reinforced Polymer (FRP); Impact loading; Energy absorption; Impact resistance.

Introduction

Rubberized concrete has compositions, which are similar to conventional concrete except a portion of normal aggregates is replaced by recycled rubber aggregates. There are many studies on rubberized concrete and they proved its many advantages comparing to conventional concrete, e.g., lightweight, green, and energy-absorption capacity [1-5]. When considering a light-weight material, rubberized concrete mixed with chipped rubber has a lower density than concrete mixed with crumb rubber and conventional concrete [6]. The density of rubber is significantly lower than conventional coarse/fine aggregates and sand so that more rubber content leads to more reduction in the self-weight of rubberized concrete. According to Elchalakani [2], the density of the rubberized concrete reduced from 2450 kg/m^3 for 0% rubber content to 1950 kg/m^3 for 40% rubber content.

In addition, rubberized concrete showed excellent performance in resisting impact loads. The excellent energy absorption of rubberized concrete was proven by previous studies [7, 8]. The rubberized concrete with excellent characteristic of good energy dissipation capacity is suitable for the applications when the energy dissipation capacity is of more importance than strength, i.e. pedestrian blocks [9], rock-fall barriers [10-14] and roadside barriers [2, 7, 15]. It was reported that the roadside barriers made of rubberized concrete had good performance and energy absorption capacity against impact loads [2, 7, 15]. It is noted that an innovative design of a cushion layer can significantly improve the impact resistance [10-14]. For the rock-fall barriers, studies on the impact-resistance behaviors of the RC barriers without cushion layers have been conducted [13, 14] and the effect of cushioning on the barriers with various cushion layers such as granular materials and gabion [10-12] has been also investigated. It was reported that using cushion layers placed in front of the barrier is effective in attenuating impact force, reducing transmitted loads to the barrier and dissipating impact energy from boulder impacts. The effectiveness of mitigation can be quantified by the cushioning efficiency i.e. load-reduction factor. The impact forces and transmitted loads to the barrier can be reduced by 25% and 50%, respectively when using cushion layer [11]. The inertial resistance of the barrier also contributes to its impact resistance. Meanwhile, the quasi-static mechanical properties and structural performance of rubberized concrete are relatively better understood than their dynamic counterparts. Many studies have been conducted on the quasi-static properties of rubberized concrete [16]. However, very limited studies have been conducted on the dynamic properties of rubberized concrete [17]. Pham et al. [18] experimentally investigated the lateral impact performance of the columns made of rubberized concrete. The findings from that study have shown that the energy absorption capacity of the column increased significantly by using rubberized concrete under impact loading. For instance, the impact energy absorption capacity of the column was increased by 58% and 63% when the rubber content increased from 0% to 15% and 30%, respectively. The rubberized concrete columns experienced nearly double displacements than the reference column prior to failure. The displacement response is even larger when they were confined with basalt fiber-reinforced polymer (BFRP).

The rubberized concrete was tested by using different axial impact rigs to study its energy absorption capacity. Gupta et al. [19] reported the impact resistance capacity of rubberized concrete with 25% rubber fiber replaced by using drop-weight rig. It was reported that the energy absorption improved considerably with the increase of rubber content. Donga et al. [20] also carried out tests on rubberized concrete to examine its impact resistance and reported that the impact resistance capacity of concrete can be increased by up to 60% through adding rubber. Atahan and Yücel [21] carried out impact tests by using Instron machine. This machine can monitor energy absorption, impact velocity, and impact force. The testing results exhibited that the rubberized concrete can effectively reduce peak impact forces and increase impact duration. Moreover, Liu et al. [17] conducted SHPB test on rubberized concrete to examine its dynamic properties . It was found that rubberized concrete had lower increment of dynamic increase factor (DIF) of strength as compared to traditional concrete. With the higher rubber content (within 10%), the rubberized concrete exhibited the increased energy absorption capability. However, when the rubber content is over 10%, the rubberized concrete showed decreasing energy absorption capacity with the increase of rubber replacement.

To further examine the impact performance of rubberized concrete, experimental tests were carried out on a structural scale, i.e. Sukontasukkul et al. [22] proposed a cushion layer made of rubberized concrete, which can be used for the bulletproof panels. The panels consist of two layers i.e. soft rubberized concrete layer and steel fiber reinforced concrete layer. The softer layer worked as a sacrificed layer in protective structures and less impact energy is transferred to the harder layer. Atahan and Sevim [7] carried out impact tests on full-scaled rubberized concrete roadside barriers by using a real vehicle with a 500 kg total mass. It was also reported that the concrete with higher rubber content had higher energy absorption capacity. Additionally, the acceleration and impact force induced in an impact event can be significantly reduced owing to the hardness of rubberized concrete, which can mitigate injury risk. Pham et al. [18] found that rubberized concrete columns could be used to effectively minimize the peak impact force up to 40% for tested specimens with 30% rubber content after comparing with the reference concrete specimen. In the latter study, the columns made of rubberized concrete with different rubber contents were subjected to lateral impact loads. Pham et al. [23] also conducted axial tests on rubberized concrete by using instrumented drop-weight impact test apparatus. The main concern of that study was the impact energy absorption and the impact force. The authors also found that the peak impact force can be mitigated up to 50% and the impact duration can be considerably extended by using rubberized concrete. As a result, they recommended that the possible use of rubberized concrete for roadside barriers might reduce injury risk to drivers and passengers of the colliding vehicle.

As can be seen from the above review, the vast majority of previous studies on rubberized concrete focused on the material scale with only a few studies that investigated the performance of beams and columns made of rubberized concrete having been reported. None study has been carried out to investigate the flexural behavior of rubberized concrete beams against impact loads. Therefore, the impact behavior of rubberized concrete beams is investigated in this study by using the drop-weight tests. It is noted that rubberized concrete provides high-energy absorption capability but it has lower strength as compared to normal concrete. Accordingly, these rubberized concrete beams were strengthened by using fiber-reinforced polymer (FRP) to enhance their load-carrying capacity for better design of beams to reduce impact force through drop-weight and beam interaction, enhance energy absorption capacity, and possess sufficient load-carrying capacities.

Impact mechanism of concrete beams under impact loads

Since the impact behaviour of rubberized concrete beams has not been investigated and reported in the literature, the impact behaviour of normal concrete beams is reviewed instead to provide a general view of its dynamic response. There were a few studies on the performance of reinforced concrete (RC) beams or FRP strengthened RC beams under impact loads [24-30]. The behaviour of RC beams under impact loads was completely different from that under static loads. The different behavior of RC beams under impact loads can be explained by two phenomena, namely the localized/global responses of the beams and shear-dominant failure mechanism. The localized/global response causes differences in the negative bending moment and reaction force while the shear-dominant mechanism relates to the inertial resistance of the beams.

Under static loading, a simply supported RC beam subjected to three-points bending shows positive moment at midspan and the whole beam contributes to resist the applied load. However, a simply supported RC beam under dynamic loading experiences positive bending moment at the midspan and negative bending moment close to the supports [28, 31, 32]. The beam also experiences negative reaction forces at the supports, which definitely do not appear under static loads. The existence of negative bending moment and reaction forces was discussed in the previous studies [28, 31, 32]. The negative reaction forces were observed in all the simply supported beams under impact tests in the previous study [28]. Pham and Hao [28] reported this interesting phenomenon, which was also confirmed by other studies [33, 34], and explained this phenomenon by the stress wave theory. The authors mentioned that the stress wave propagations caused this phenomenon. Therefore it is a stress wave propagation problem rather than a structural mechanics or a structural dynamics problem. The surface Rayleigh wave, which accounts for 67% impact energy, may cause this phenomenon. Meanwhile, the negative bending moment in the simply supported beams under impact loads was observed experimentally in the tests and numerically simulated in the previous study by Pham et al. [35]. The experimental tests clearly showed some vertical cracks on top of the beam at the negative bending moment region. Accordingly, similar damage and cracks at the negative bending moment region were seen in the numerical simulation. The authors explained the formation of the negative bending moment was caused by the local response of the beam. When a simply supported beam is impacted by a projectile, only a portion at the midspan of the beam responds [35, 36] during the early stage of the impact. The two beam ends and boundary conditions do not affect the responses at the initial stage of the impact. Accordingly, the negative bending moment only occurs in a relatively long beam while short beams do not show this phenomenon as clarified in the previous study by Pham and Hao [32].

Unlike static behaviour, the impact behaviour of RC beams includes two phases including the impact loading phase and the free-vibration phase. In the impact loading phase, which happens in a very short period of about a few milliseconds, the impact force at the impact point quickly reaches to the peak value associated with localized damage at the impact area and inclined shear cracks. The beams may fail with shear dominant failure mechanism even though they would have failed in a flexural mode and a ductile manner under static loads. For instance, Pham et al. [35] tested two identical RC beams, in which Beam 1 failed with a pure flexural manner under static load with only vertical cracks and Beam 2 was damaged with a mixed shear-flexure mode when subjected to impact load. It is noted that the two tested beams were designed to have the shear capacity four times the flexural capacity so that it would experience pure flexural failure as also observed in the static tests. Interestingly, the shear failure was observed for the beam under impact loads. This phenomenon was identified as the shear-dominant mechanism of RC beams under impact loads as mentioned in the previous studies [28, 37]. The previous studies also examined and explained the shear mechanism of structures subjected impact loads, which is the most common failure mode [37-39].

In general, the impact behaviour is governed by many parameters in which the peak impact force plays an essential role as shown in the previous study by Do et al. [40]. The authors demonstrated that under vehicle-column collision, the peak impact force might change the column failure mode even though the impact force time histories of various scenarios are almost similar (except the peak impact force). The impact force is influenced by the contact stiffness between the projectile and the beam at which the hardness and modulus of the material and contact quality are critical parameters as discussed in the previous study by Pham et al. [35]. The rubberized concrete has lower modulus so that it results in lower peak impact force under similar impact energy [23]. Accordingly, the inertia resistance will be lower and thus the impact behaviour of the rubberized concrete is distinguished from that of normal concrete. These variations are experimentally investigated in this study.

Experimental program

Specimen design and material properties

Twelve rubberized reinforced concrete (RuC) beams were cast and they had dimension of 2200 mm in length by 250 mm in height by 150 mm in width. These beams included 6 reference beams without rubber, 3 beams with 15% rubber content, and 3 beams with 30% rubber content (Table 1). Among these beams, nine beams were wrapped with different schemes as shown in Fig. 1. The beams (Bxy_zm) are labelled based on their rubber contents, strengthening scheme, and drop height. For example, Beam B15A_2m means this beam was made of 15% rubberized concrete, strengthened with scheme A, and tested under 2 m drop height. The rubber content indicates the amount of rubber aggregates used to replace 15% or 30% of the total volume of the aggregate. N10 and N12 steel rebars were used as longitudinal reinforcements with a length of 2160 mm. Steel stirrups had a dimension of 10 mm in diameter and a spacing of 115 mm.

The rubber aggregates included crumb rubber (7-10 mm) and chip rubber (2-5 mm), which was pre-treated by 10% of NaOH solution. This is a crucial procedure as it significantly improves the bonding between rubber aggregates and the cement matrix. The treatment process was carried out as follows: (1) thoroughly washed the rubber aggregates to remove all the impurities and dust which might weaken the bonding strength between rubber aggregates and the cement matrix, (2) prepared 10% of NaOH aqueous solution and used it to soak rubber particles for a minimum duration of 24 hours, (3) drained the NaOH solution and the rubber aggregates were rinsed with water to achieve a pH of 7, (4) dried the rubber aggregates under the sun before the actual use.

The unidirectional BFRP sheets used in this experiment had the dimensions of 100 mm in width and 0.12 mm in thickness and a density of 300 g/m^2. It has the properties of 2100 MPa in tensile strength, 77.9 GPa in elastic modulus, and 2.1% in tensile elongation as reported in the previous study [41]. Prior to bonding BFRP, surface preparation was carefully conducted by removing weak concrete. Air gun was used to blow the concrete surface and remove all dust and weak concrete. More details about the surface preparation can be found in previous studies [28, 29]. Epoxy resin including two parts with a mixing ratio at 5:1 was used [42]. The tensile strength, modulus, and elongation of the epoxy resin were 54 MPa, 2.8 GPa, and 3.4%, respectively. These beams are divided into three groups based on their BFRP bonding schemes. The first group will be the reference beams with no BFRP strengthening. The second group will have beams with four longitudinal BFRP strips as illustrated in design A in Figure 1, while the third group will have beams with four longitudinal BFRP strips and one 45° inclined U-shape wraps as shown in design B in Figure 1. The classification of each beam specimen and their respective tests are summarized into the test matrix as shown in Table 1. Strain gauges (SG) were bonded to BFRP wraps to monitor their strain in the longitudinal and transverse directions. There were two SGs on the longitudinal BFRP sheets including one at the midspan and another one offset 250 mm from the beam end. Another strain gauge was attached to the middle of the 45^o inclined U-shaped layer close to the midspan.

Impact Testing Procedure

All the beams were tested under 2 m drop height for the first drop. If the beams did not fail, they were tested again under 2.5 m and then repeated until failure. An instrumented drop-weight test system, which releases a weight from a designated height onto the midspan of the beams, was used for all the impact tests as shown in Fig. 2. The drop weight weighs 208.8 kg. The shape of the impactor had a smooth spherical bottom (50 mm radius). The impactor vertically drops to specimens by using a plastic tube. The boundary condition was carefully designed to achieve the simply supported beams, in which the supports were restrained in the vertical direction to prevent rebounding of the beams. Two upper and lower load cells with the capacity of 25 ton each were fixed to two sides of the beam supports as shown in Fig. 3, which were used to record the positive and negative reaction forces. The effective span of these beams was 1900 mm, which was created by steel rollers. A steel load adaptor (100 x 100 x 20 mm) was fixed on the top surface of the beams at the centre and the impact load cell (180 ton) was bolted to the load adaptor. A high-speed camera was used to monitor the failure processes of these beams and it was set as 20,000 frames per second. The data acquisition system was utilized to record the signals of the FRP sheets and load cells from strain gauges. The sampling rate of data acquisition system was set as 100 kHz. Signals of the impact forces were filtered by FFT low-pass with a cutoff frequency of 10 Hz.

Experimental results and discussions

Crack development and failure modes

All the beams were tested under a drop height of 2 m for the first hit. The drop height increased to 2.5 m for the second hit if the beam did not fail under the first impact. Damage at the impact area is larger than the load distributor and this is similar to the damage of the front face of a concrete panel subjected to projectile impact [43]. The longitudinal compressive stress waves generated by the impact spherically propagated into the beams. The spalling of concrete at the two sides of the beam, close to the impact area, was observed as shown in Fig. 4. The spalled concrete debris was still intact after failure. The tensile spalling of concrete at the two sides of the beams was also reported in previous studies [28, 29]. When the stress wave arrived at the free surface of the beams at the side, it was reflected as a tensile stress wave. The original compressive stress wave interacted with the reflected tensile stress wave resulted in a decreasing compressive wave but increasing tensile wave amplitude. The increasing tensile stress wave might exceed the dynamic tensile strength of concrete, which might generate cracks in the beam as shown in Fig. 5. The tensile stress wave propagated to the free surfaces of the beams and reflected back as a compressive stress wave. This process was repeated and might have caused further damage to the beams until the resultant stress waves were lower than the dynamic tensile strength of concrete.

Failure modes were associated with the shear dominant mechanism as can be seen from the shear plug at the impact area, in which the shear plug indicates a trapezium concrete region underneath the impact point. The reference beam without rubber content showed distributed flexural and shear crack along the beam axis. Meanwhile, the rubberized concrete beams showed more localized damage because of the shear dominant mechanism under impact tests, which was explained clearly in previous studies [28, 37]. The shear resistance of the beams becomes more critical under impact tests at which the rubberized concrete beams had much lower shear resistance so that they failed in a shear-govern mechanism. This phenomenon was clearly observed in beam B30_2m at which cracks only appeared within the shear plug under the impact area. The BFRP strengthened beams showed similar crack map regarding the corresponding unstrengthened specimens with a combination of flexural and shear cracks. However, BFRP sheets prevented crack development in the strengthened beams so that they failed under higher impact energy with rupture of the longitudinal BFRP sheets and partial debonding of vertical/inclined U-shaped BFRP strips as shown in Fig. 5.

In addition, the flexural cracks induced by negative bending moments were observed in these tests as shown in Fig. 5. Under static tests, it is obvious that the simply-supported beam under point load only generates positive bending moment along the beam. However, a simply supported beam may induce a negative bending moment on the beam top close to the supports. The formation of the negative bending moment was reported and discussed in the previous studies [31, 32, 34]. This negative bending moment caused flexural cracks in the upper surface as shown in Fig. 5. The cracks at the negative bending moment area were caused by the inertial resistance of the beams. The higher impact velocity leads to high acceleration of the beams and thus causes higher inertia resistance, which leads to a higher negative bending moment and thus more cracks at the negative bending area. This phenomenon is shown in Fig. 5 where the beams under 2.5 m drops yielded more cracks at the negative bending moment area. This observation again confirmed the existence of the negative bending moment and its consequences so that it needs to be taken into consideration when designing concrete beams against impact loads.

Dynamic response

The impact response of the tested beams included two continuous stages, which are the impact force transient phase and the free vibration phase. The impact force phase occurred in about 10 ms in which the peak impact force last for approximately 1 ms while the reaction force last much longer up to 10 ms. Afterward, the beams exhibited a free vibration phase in which the beams freely vibrated and damped after more than 100 ms. The time histories of the impact force and midspan displacement are presented and discussed in the following sections. The time histories of the impact force and the reaction force of the unstrengthened beams are shown in Fig. 6. The use of rubberized concrete reduced the peak impact forces and reaction force. Unfortunately, the impact force time histories of Beam B30_2m was not recorded due to a malfunction in the data acquisition system. When the rubber content increased, the reduction of the impact forces and reaction forces under the same impact energy was observed as shown in Fig. 7. The maximum impact forces of Beams B0A_2m and B30A_2m were 1800 kN and 1500 kN, respectively. The maximum reaction forces of these two beams were 52 kN and 40 kN, respectively. These figures clearly demonstrate that using rubberized concrete can considerably reduce the impact forces and reaction forces in the beams. The reduction of the impact forces was also observed for other BFRP strengthened rubberized concrete beams as compared to the conventional concrete beams. Meanwhile, the duration of the peak impact forces of the rubberized concrete beams was longer than that of the conventional concrete beams, for example the duration of the peak impact forces of Beam B0A_2m and B30A_2m was 0.6 ms and 1 ms, respectively.

In the meantime, another unique phenomenon associated with the impact behaviour of a simply supported beam, which is the existence of the negative reaction force earlier than the positive one, was also observed in the tests as shown in Figs. 6-8. This phenomenon was reported in previous studies [28, 33, 34]. Pham and Hao [32] attempted to explain this interesting phenomenon by using the theory of stress wave propagation. Upon an impact event on a solid surface, P-wave, shear wave and surface Rayleigh wave dissipated 7%, 26% and 67% of the impact energy, respectively [44]. Both the P-wave and shear wave propagate faster than Rayleigh wave and diminish at a faster rate because the former ones have higher-frequency contents. In these concrete beams, P-wave arrives the supports first and then shear wave while Rayleigh wave comes last. Shear wave and P-wave cause the longitudinal and transverse vibrations of the beam, respectively. Since vibrations in the horizontal directions, caused by both P-wave and shear wave, do not generate vertical loads so that these two waves cannot be monitored by the load cells fixed in the vertical direction. Therefore, the reaction force in the negative direction was likely caused by the arrival of Rayleigh wave. All the beams in this study showed the magnitude of the negative reaction force was about a half the corresponding positive ones (20-25 kN vs 40-50 kN).

Additionally, the time histories of displacement are traced from the high-speed camera videos and shown in Fig. 9. It is interesting that the displacement of Beams B15 and B30 was lower than that of conventional concrete beam B0, the rubberized concrete beams deformed slightly less (43.5 mm vs 45.5 mm, 4.4% reduction) than that of the reference beam under impact loads. Meanwhile, the displacement at maximum static loads can be reasonably estimated by using the software Response 2000 [45], such as the displacement at maximum static loads of Beams B0, B15 and B30 is 16.8 mm, 19.2 mm, and 21.6 mm, which shows an increase of 14.3% and 28.6%, respectively. This observation demonstrates that the rubberized concrete beams possess an excellent performance under impact loads. In addition, the maximum displacement of Beam B30 was slightly higher (44.5 mm vs 43.5 mm) than that of Beam B15 so that it suggests that the optimized rubber content is close to 15%. This observation is valid for all other strengthening schemes since the beams with 15% rubber content always showed lower maximum displacement as compared to other corresponding beams as shown in Fig. 9. In addition, replacing normal aggregates by rubber aggregates did not considerably change the vibration characteristics of the beams. Fig. 9 shows that the natural vibration period of Beams B0, B15 and B30 was 24.9, 24.2, and 23.6 ms, respectively. To verify the measured natural vibration period, the predicted natural vibration period was estimated based on the material properties and the beams' dimensions. The mix design of the rubberized concrete in this studies was adopted from the previous studies [6, 18]. The Young's modulus of RuC with the rubber contents of 15% and 30% were estimated as 19 GPa and 15 GPa, respectively [2]. The natural circular frequency of the beams is estimated as follows [46]:

$$\omega = \sqrt{\frac{k}{m^*}} \tag{1}$$

where k and m^* are respectively the stiffness and effective mass of the beam, taken as $0.493m$. It is noted that m is the mass of the beam within the effective length, L_e, and k is estimated as follows:

$$k = \frac{48EI}{L_e^3} \tag{2}$$

where E is the elastic modulus and I is the moment of inertia of the beam which considers cracked section. The natural period of vibration T is estimated as follows:

$$T = \frac{2\pi}{\omega} \tag{3}$$

In the impact force transient phase, only a portion of the beam responds to the impact. However, the entire beam vibrates to the impact in the free vibration stage. Therefore, the effective length is considered as the whole length in this analysis. For an approximation, the moment of inertia of RC beams associated with a cracked section is about 35% of the uncracked section [47]. The estimated natural period of vibration of the beams with 15% and 30% rubber content is 19 ms and 21 ms, respectively. The measured period of vibration of these beams was 24.2 and 23.36 ms, respectively. This variation is reasonable since the modulus and the stiffness of the crack section was approximately estimated for the rubberized concrete beams. This verification indicates the reliability of the testing results. Meanwhile, from the displacement time histories of all the tested beams, it can be seen that the damping ratio of the strengthened beam was much higher than that of the unstrengthened beams. The vibration of the first ones vanished after only one or two cycles while the later ones damped after more than 4 cycles.

Time lag and stress wave velocity

The impact force time histories from the tested beams show that there was a delay in the arrival time between the impact force at the midspan and the reaction force at the supports, called time lag. It might be estimated from the initiation of the impact force as well as the reaction force [28], which is affected by the stress wave velocity. The time lag between impact and reaction forces of the tested beam shown in Fig. 10 indicates the time lag was affected by the rubber content and damage of concrete. Under the first impact of 2 m, there were no existing cracks in these beams. The time lag for beams with 0%, 15%, and 30% rubber content was 0.52 ms, 0.55 ms, and 0.70 ms, respectively. This time lag is related to the required time for stress waves propagate from the impact point at midspan toward the two supports. In this study, the stress wave velocity of these beams was 1827 m/s, 1727 m/s, and 1357 m/s, respectively. It is evident that increasing the rubber content from 0% to 15% only slightly reduced the stress wave velocity by 5% while increasing the rubber content to 30% significantly decreased the stress wave velocity by 26%. These stress wave velocities can be theoretically estimated from the velocity of P-wave ($c=\sqrt{(E/\rho)}$), which are 3290 m/s, 2887 m/s, and 2630 m/s. From both the measured and estimated values, the stress wave velocity decreases with an increase of the rubber replacement. However, the measured stress wave velocity was almost half of the estimated ones. These variations were stated in previous studies [28, 34, 48]. Pham and Hao [32] suggested that the measured stress wave from the impact point toward the boundaries, which is located on the surface of the beams, should be the velocity of R-wave as discussed previously. Rhazi et al. [44] reported the velocity of R-wave in concrete is approximately 2100-2500 m/s. The theoretical value of R-wave velocity is still slightly higher than the measured velocity. The reason for this difference was clarified in the previous study [28], i.e. the load cells and steel plates in the test setup make the actual travel distance of the stress wave greater than 0.95 m. The actual stress wave velocity in the concrete should be higher than the measured values but it is close to the theoretical value for R-wave.

In addition, Fig. 10 also shows the time lag of the tested beams under different drop heights. If a beam survives under 2m-drop, it was tested again under 2.5m-drop. As a result, these beams had some pre-cracks before being tested under 2.5 m. Concrete in these pre-cracked beams obviously had a lower modulus and thus lower stress wave velocity. The measured time lag of the beams with 0%, 15% and 30% rubber content under second impact was 0.82 ms, 0.86 ms, and 1.00 ms, respectively. These values correspond to the wave velocity of 1158 m/s, 1104 m/s, and 950 m/s, respectively. Repeatedly, the stress wave velocity reduces with an increase in the damage to the tested beams. The level of reduction was also dependent on the rubber contents. However, increasing the rubber content from 0% to 15% only leads to a minor change (<5%) in the stress wave velocity while the stress wave velocity in beams with 30% rubber showed a reduction of 18% in the second drop, less than the 26% reduction during the first drop, compared to the reference beam.

Effectiveness of longitudinal and U-wrap BFRP

The rubberized concrete beams reduced the peak impact forces and had more pronounced localized damage at the impact area as shown in Fig. 5. To improve the load carrying capacity of these beams, longitudinal and transverse BFRP strips were used. As shown in Table 2 and Fig. 9, the use of BFRP strengthening significantly reduced both the maximum and residual displacements. For example, the maximum and residual displacements of Beam B0_2m were 45.6 mm and 32.9 mm. These values for beams B0A_2m and B0B_2m were 33.4 mm, 33.4 mm, 16.1 mm, and 12.2 mm, respectively. The use of BFRP strengthening reduced the maximum and residual displacement approximately by 27% and 56%, respectively. As expected, strengthening scheme B with the combination of longitudinal BFRP sheets and transverse BFRP sheets yielded better results. The effectiveness of using BFRP strengthening for the rubberized concrete beams was similar to that of the conventional concrete beams as given in Table 2. Therefore, it can be concluded that the BFRP strengthening efficiency was similar for both conventional and rubberized concrete beams.

Even though the use of BFRP strengthening greatly reduced the displacement of the beams, it exhibited a minor effect on the maximum impact force and impact force duration as given in Table 2. An increase of about 10% was observed in the maximum impact force when comparing the peak impact force of the strengthened beams with the corresponding unstrengthened ones. Meanwhile, the substantial reduction in the maximum and residual displacement of the strengthened beams indicated that these beams had higher global stiffness than the unstrengthened beams. This phenomenon shows that the increase in the global stiffness marginally affected the impact force. This observation was theoretically and numerically explained in the previous study by Pham and Hao [36], which concluded that the local stiffness governs the peak impact force while the global stiffness controls the maximum displacement.

As mentioned previously that the shear dominant mechanism affects the impact force and the dynamic resistance capacity of the tested beams. Therefore, using U-shape BFRP strips to locally improve the shear resistance capacity at the impact region may be a good design rather than uniformly distributing the U-shape BFRP strips along the beams. Fig. 11 shows the impact force time histories of the four beams with 0% rubber content and different strengthening schemes (A, B, C, and D). Beam B0A_2m exhibited the lowest maximum reaction force (51.7 kN) while the corresponding values of Beams B0B_2m, B0C_2m, and B0D_2m were 60.3 kN, 66.8 kN, 55.3 kN, respectively. In terms of reaction forces, these beams with U-shape BFRP strips showed higher reaction forces to that of Beam B0A_2m, which only had longitudinal BFRP strips. As can be seen from Fig. 11, Beam B0C_2m which had only a half number of U-shape BFRP strips compared to that of Beam B0B_2m but the former one even shows slightly higher impact force and reaction force. However, this difference is minor and can be considered as variation in testing. From this observation, it can be concluded that locally strengthening a beam in shear with U-shape BFRP yields similar impact resistance as beams which had uniformly distributed U-shape BFRP strips. This phenomenon is different from the static case when the beam with uniformly distributed BFRP U-shape wraps resisted higher loads and deformed at a much higher displacement as reported by Chen et al. [41]. To further investigate the impact response of these beams, the displacement time histories of these beams are shown in Fig. 12. It is seen that the displacement time history of Beam C is similar to Beams B and D. The maximum and residual displacements of these beams were in the range of 31-33 mm and 11-16 mm, respectively. This observation suggests that strengthening beams at the impact region by using vertical wraps is very effective and can provide a cost-saving solution.

Imparted Energy

The impact energy is reversed back into the rebound in an ideally elastic impact while a portion of the impact energy is imparted in elastic deformation and another part of the impact energy is consumed in the plastic deformation and failure in real impacts [49]. The energy-balanced method can be used to equate the input kinetic energy and the component energies in a beam. The energy-balanced equation can be expressed as follows:

$$\frac{1}{2}M\left(V_1^2 - V_2^2\right) = E_b + E_s + E_m + E_c + E_k \tag{4}$$

where M is the projectile weight, V_1 and V_2 are the initial impact velocity and residual velocity of the projectile, respectively, E_b, E_m, E_s, E_c represent the energy in the form of bending deformation, membrane component, shear deformation, and indentation effect when the projectile rebounds from the beam, respectively, and E_k is the kinetic energy of the beam. After separating from the projectile, the beam further deforms and reaches its maximum displacement when the kinetic energy in the beam equals zero. From the energy conservation law, the energies in the beam can be equated as follows:

$$\frac{1}{2}M\left(V_1^2-V_2^2\right)=E^*{}_b+E^*{}_s+E^*{}_m+E_c \tag{5}$$

where $E^*{}_s$, $E^*{}_b$, and $E^*{}_m$ are energies representing the shear deformation, bending deformation, and membrane component when the beam reaches the maximum displacement, respectively. It is noted that the energy caused the local indentation does not change and the kinetic energy of the beam vanishes at the maximum displacement. Since the stretching effect is small and can be ignored in the beam behavior, the energy for the membrane effect can be excluded [50].

The energy-balanced method was discussed and adopted to predict the impact response and impact forces in previous studies [50-54]. Predictions from the proposed models in these studies matched the experimental results quite well. The models were derived based on two assumptions, which need to be carefully justified. Firstly, structures were assumed to behave in a quasi-static manner, at which the structures reach their maximum displacement when the beam velocity becomes zero [51, 53]. Secondly, the energies are derived based on the load-displacement under quasi-static loads [50, 54]. Zhou and Stronge [54] recommended their model was based on the static behavior, thus, it is only meaningful in the case of a heavy projectile impacting a light plate. The two assumptions are not necessarily correct in the impact tests in this study. In all the impact force time histories in this study, the impact forces ceased at about 1-2 ms while the beams reached the maximum displacements at approximately 20 ms. As a result, the separation between the projectile and the beam occurred before the beams reached the maximum displacement. In addition, estimating the energies by using the bending and shear stiffness of the beams usually adopts the global stiffness for the whole impact duration. However, it has been proven in previous studies that only a portion of a beam reacts to the impact force in the force phase (for the case of impact tests on concrete beams) and the effective span of the beam is shorter than its actual span [31, 32, 34, 36, 48, 55]. This observation is because the beam locally responds to the impact loads, instead of globally deforms as in a static case. In the previous study, Abrate [51] concluded that using the energy absorption from the assumption of a static case is inappropriate in this case because the impact force vanishes when the beam deflection has not reached its maximum value. To distinguish whether the local response or global response governs the beam behavior, the ratio between the impact loading duration, t_d, and the structural vibration period, T, was used.

There is an alternative way to quantify the energy imparted into the beams by using the variation of the kinetic energy of the projectile as presented in Eqs. 4-5. The imparted energy of the tested beams is given in Table 2. For unstrengthened beams, the imparted energy of the rubberized concrete beams was slightly less than that of the reference beams with a reduction of 1.9% and 8.1% for Beams B15_2m and B30_2m, respectively. However, these beams have different masses so that it is more useful if the imparted energy per unit weight is examined (Table 2). The imparted impact energy per unit weight of the rubberized concrete was significantly higher that of the reference beam (B0_2m) with an increase of 10.3% and 17.8% for Beams B15_2m and B30_2m, respectively. Meanwhile, strengthening the beams with BFRP sheets did not increase the energy imparted to the beams as shown in Fig. 13. Meanwhile, when higher impact energy is applied to the beams, more impact energy imparted to the beams. Strengthening schemes A and B did not show a difference in the imparted energy in the conventional concrete beams. However, strengthening scheme B clearly show higher imparted energy than that of strengthening scheme A for the rubberized concrete beams, particularly under 2.5-m drops. In general, the imparted energy of the strengthened rubberized concrete beams was almost similar to the corresponding conventional concrete beams under 2 m drops but they exhibited higher imparted energy than the reference beams under 2.5 m drops.

Strain and failure of BFRP

BFRP strain in the longitudinal direction at the midspan was measured by strain gauges and plotted in Figs. 14-15. Fig. 14 shows BFRP strain of the tested beams under 2 m drop, in which the maximum BFRP strain was about 1% which was comparable to the results under static tests reported in previous studies [56, 57]. BFRP strain quickly increased to the peak values and then reduced to the plateau value of 0.5-0.6% for all the beams, except for Beam B30B_2m. Unfortunately, BFRP strain of some beams was lost due to a malfunction in the data acquisition system. All the beams did not show debonding of longitudinal BFRP sheets under 2 m drops. These beams were then tested and failed under 2.5 m drop height. The beams strengthened with scheme A failed by intermediate crack induced debonding of the bottom BFRP sheets close to the midspan and then propagated toward the supports. The two major shear cracks under the impact region developed at a very early stage and initiated the debonding. Meanwhile, the rupture of BFRP sheets lead to failure of the strengthened beams with scheme B (Fig. 16). As a result, the maximum BFRP strain of beams type B was greater than that of beams type A, for instance, the maximum BFRP strain of Beams B15B_2.5m and B30B_2.5m was 0.6% and 0.65%, respectively, while the corresponding numbers of Beams B15A_2.5m and B30A_2.5m were 0.46% and 0.45%, respectively (Fig. 15). It is worth mentioning that the BFRP strain under the second impact of 2.5m was measured on the top of the residual BFRP strain as shown in Fig. 14 so that the actual BFRP strain at failure should be a sum of these values. The relatively high values of BFRP strain prove that it can be effectively used to strengthen concrete structures against impact without losing its effectiveness as reported in previous studies. The cohesive debonding of BFRP in beams type

A, associated with a thin layer of concrete, exhibited sufficient bond strength between BFRP and concrete (Fig. 16), which implies good workmanship.

Conclusions

This study experimentally examines the impact behaviour of RuC beams strengthened with BFRP. The experimental results clearly show that the rubberized concrete beam deformed less than the conventional concrete beam under the same impact energy even though they had lower compressive strength of concrete and static strength. The findings can be summarized as follows:

1. Rubberized concrete had 10-18% higher imparted energy per unit weight than that of normal concrete.
2. Under the same impact energy, rubberized concrete reduces the maximum impact force while providing slightly less deformation relative to conventional concrete.
3. BFRP strengthening sufficiently improves the impact resistance of the beams made of both conventional and rubberized concrete. The BFRP strengthened beams damped the impact force at a faster rate than the corresponding unstrengthened ones.
4. Locally strengthening the beams with U-shape BFRP wraps at the impact points generates the same impact performance as compared to using U-shape BFRP wraps for the entire beams but with a cheaper cost.
5. The debonding strain of BFRP wraps under impact tests was similar to that under static tests so that it can be used efficiently to strengthened RC beams against impact loads.

Finally, rubberized concrete is a green and lightweight material. With a much lower compressive strength, its impact performance is slightly better than conventional concrete with lower maximum impact force and displacement and similar imparted energy.

Acknowledgement

The authors acknowledge the financial support from Australian Research Council (ARC LP150100259). The authors would like to deeply thank Adrian Jones from Tyrecycle. Supports from Andrew Sarkady and Anup Chakrabortty are appreciated. The technical supports from Jim Waters and Brad Rose are acknowledged. Thanks are given to Mohd Zobair Nazifa, Muaz Ibrahim, and Yung Sheng Lee for performing the tests.

References

[1] Gupta T, Chaudhary S, and Sharma RK. Assessment of mechanical and durability properties of concrete containing waste rubber tire as fine aggregate. Constr Build Mater 2014;73:562-74.

[2] Elchalakani M. High strength rubberized concrete containing silica fume for the construction of sustainable road side barriers. Structures 2015;1:20-38.

[3] Elchalakani M, Aly T, and Abu-Aisheh E. Sustainable concrete with high volume GGBFS to build Masdar City in the UAE. Case Studies in Construction Materials 2014;1:10-24.

[4] Elchalakani M, Hassanein M, Karrech A, and Yang B. Experimental investigation of rubberised concrete-filled double skin square tubular columns under axial compression. Eng Struct 2018;171:730-46.

[5] Elchalakani M, Hassanein M, Karrech A, Fawzia S, Yang B, and Patel V. Experimental tests and design of rubberised concrete-filled double skin circular tubular short columns. Structures: Elsevier; 2018. p. 196-210.

[6] El-Sherbini Y, Abdel-Gawad A, Shalaby A, and El-Gammal A. Compressive strength of concrete utilizing waste tire rubber. Journal of Emerging Trends in Engineering and Applied Sciences 2010;1:96-9.

[7] Atahan AO, and Sevim UK. Testing and comparison of concrete barriers containing shredded waste tire chips. Mater Lett 2008;62:3754-7.

[8] Ozbay E, Lachemi M, and Sevim UK. Compressive strength, abrasion resistance and energy absorption capacity of rubberized concretes with and without slag. Mater Struct 2011;44:1297-307.

[9] Sukontasukkul P, and Chaikaew C. Properties of concrete pedestrian block mixed with crumb rubber. Constr Build Mater 2006;20:450-7.

[10] Ng C, Choi CE, Su A, Kwan J, and Lam C. Large-scale successive boulder impacts on a rigid barrier shielded by gabion. Can Geotech J 2016;53:1688-99.

[11] Ng C, Su Y, Choi C, Song D, Lam C, Kwan J, Chen R, and Liu H. Comparison of cushioning mechanisms between cellular glass and gabions subjected to successive boulder impacts. Journal of Geotechnical and Geoenvironmental Engineering 2018;144:04018058.

[12] Bourrier F, Nicot F, and Darve F. Physical processes within a 2D granular layer during an impact. Granular Matter 2008;10:415-37.

[13] Yong AC, Lam C, Lam NT, Perera JS, and Kwan JS. Analytical Solution for Estimating Sliding Displacement of Rigid Barriers Subjected to Boulder Impact. Journal of Engineering Mechanics 2019;145:04019006.

[14] Majeed ZZ, Lam NT, Lam C, Gad E, and Kwan JS. Contact Force Generated by Impact of Boulder on Concrete Surface. Int J Impact Eng 2019:103324.
[15] Topçu IB, and Avcular N. Collision behaviours of rubberized concrete. Cem Concr Res 1997;27:1893-8.
[16] Pacheco-Torgal F, Ding Y, and Jalali S. Properties and durability of concrete containing polymeric wastes (tyre rubber and polyethylene terephthalate bottles): An overview. Constr Build Mater 2012;30:714-24.
[17] Liu F, Chen G, Li L, and Guo Y. Study of impact performance of rubber reinforced concrete. Constr Build Mater 2012;36:604-16.
[18] Pham TM, Zhang X, Elchalakani M, Karrech A, Hao H, and Ryan A. Dynamic Response of Rubberized Concrete Columns with and without FRP Confinement Subjected to Lateral Impact. Constr Build Mater 2018;186:207-18.
[19] Gupta T, Sharma RK, and Chaudhary S. Impact resistance of concrete containing waste rubber fiber and silica fume. Int J Impact Eng 2015;83:76-87.
[20] Donga PD, Shah D, and Bhavsar JK. Impact Resistance of Waste Rubber Fiber Silica Fume Concrete. Journal of Civil Engineering and Environmental Technology 2016;3:274-9.
[21] Atahan AO, and Yücel AÖ. Crumb rubber in concrete: Static and dynamic evaluation. Constr Build Mater 2012;36:617-22.
[22] Sukontasukkul P, Jamnam S, Rodsin K, and Banthia N. Use of rubberized concrete as a cushion layer in bulletproof fiber reinforced concrete panels. Constr Build Mater 2013;41:801-11.
[23] Pham TM, Elchalakani M, Karrech A, and Hao H. Axial impact resistance of rubberized concrete with/without FRP confinement for sustainable road side barriers. Int J Protect Struct 2019;10:154-73.
[24] White T, Soudki K, and Erki M. Response of RC Beams Strengthened with CFRP Laminates and Subjected to a High Rate of Loading. J Compos Constr 2001;5:153-62.
[25] Tang T, and Saadatmanesh H. Analytical and experimental studies of fiber-reinforced polymer-strengthened concrete beams under impact loading. ACI Struct J 2005;102:139-49.
[26] Erki M, and Meier U. Impact loading of concrete beams externally strengthened with CFRP laminates. J Compos Constr 1999;3:117-24.
[27] Tang T, and Saadatmanesh H. Behavior of Concrete Beams Strengthened with Fiber-Reinforced Polymer Laminates under Impact Loading. J Compos Constr 2003;7:209-18.
[28] Pham TM, and Hao H. Behavior of fiber reinforced polymer strengthened reinforced concrete beams under static and impact loads. Int J Protect Struct 2017;8:1-22.
[29] Pham TM, and Hao H. Impact behavior of FRP-strengthened RC beams without stirrups. J Compos Constr 2016;20:04016011.
[30] Capozucca R. Assessment of CFRP strengthened RC beams through dynamic tests. Compos Part B 2013;46:69-80.
[31] Pham TM, and Hao H. Effect of the plastic hinge and boundary condition on the impact behaviour of RC beams. Int J Impact Eng 2017;102:74-85.
[32] Pham TM, and Hao H. Plastic hinges and inertia forces in RC beams under impact loads. Int J Impact Eng 2017;103:1-11.
[33] Kishi N, and Mikami H. Empirical formulas for designing reinforced concrete beams under impact loading. ACI Struct J 2012;109:509-19.
[34] Cotsovos DM. A simplified approach for assessing the load-carrying capacity of reinforced concrete beams under concentrated load applied at high rates. Int J Impact Eng 2010;37:907-17.
[35] Pham TM, Hao Y, and Hao H. Sensitivity of impact behaviour of RC beams to contact stiffness. Int J Impact Eng 2018;112:155-64.

[36] Pham TM, and Hao H. Influence of global stiffness and equivalent model on prediction of impact response of RC beams. Int J Impact Eng 2018;113:88-97.
[37] Saatci S, and Vecchio FJ. Effects of shear mechanisms on impact behavior of reinforced concrete beams. ACI Struct J 2009;106:78-86.
[38] Zhao D-B, Yi W-J, and Kunnath SK. Numerical simulation and shear resistance of reinforced concrete beams under impact. Eng Struct 2018;166:387-401.
[39] Do TV, Pham TM, and Hao H. Impact Force Profile and Failure Classification of Reinforced Concrete Bridge Columns against Vehicle Impact. Eng Struct 2019;183:443-58.
[40] Do TV, Pham TM, and Hao H. Dynamic responses and failure modes of bridge columns under vehicle collision. Eng Struct 2018;156:243-59.
[41] Chen W, Pham TM, Sichembe H, Chen L, and Hao H. Experimental study of flexural behaviour of RC beams strengthened by longitudinal and U-shaped basalt FRP sheet. Compos Part B 2018;134:114-26.
[42] West System n.d. Epoxy resins and hardeners - Physical properties. 2017.
[43] Almusallam TH, Siddiqui NA, Iqbal RA, and Abbas H. Response of hybrid-fiber reinforced concrete slabs to hard projectile impact. Int J Impact Eng 2013;58:17-30.
[44] Rhazi J, Hassaim M, Ballivy G, and Hunaidi O. Effects of concrete non-homogeneity on Rayleigh waves dispersion. Mag Concr Res 2002;54:193-201.
[45] Collins MP. Response 2000. 2000.
[46] Wu KQ, and Yu TX. Simple dynamic models of elastic–plastic structures under impact. Int J Impact Eng 2001;25:735-54.
[47] Warner RF, Foster SJ, and Kilpatrick AE. Reinforced concrete basics: analysis and design of reinforced concrete structures: Pearson Education Australia; 2007.
[48] Fujikake K, Li B, and Soeun S. Impact response of reinforced concrete beam and its analytical evaluation. J Struct Eng 2009;135:938-50.
[49] Zhang Q, Zhang J, and Wu L. Impact and energy absorption of long fiber-reinforced thermoplastic based on two-phase modeling and experiments. Int J Impact Eng 2018.
[50] Hazizan MA, and Cantwell W. The low velocity impact response of an aluminium honeycomb sandwich structure. Compos Part B 2003;34:679-87.
[51] Abrate S. Impact on composite structures: Cambridge university press; 2005.
[52] Pham TM, and Hao H. Prediction of the impact force on RC beams from a drop weight. Adv Struct Eng 2016;19:1710-22.
[53] Foo CC, Seah LK, and Chai GB. A modified energy-balance model to predict low-velocity impact response for sandwich composites. Compos Struct 2011;93:1385-93.
[54] Zhou D, and Stronge W. Low velocity impact denting of HSSA lightweight sandwich panel. International Journal of Mechanical Sciences 2006;48:1031-45.
[55] Hao H. Predictions of Structural Response to Dynamic Loads of Different Loading Rates. Int J Protect Struct 2015;6:585-606.
[56] Yuan C, Chen W, Pham TM, and Hao H. Effect of Aggregate Size on Bond Behaviour between Basalt Fibre Reinforced Polymer Sheets and Concrete. Compos Part B 2019;158:459-74.
[57] Yuan C, Chen W, Pham TM, and Hao H. Bond Behavior between FRP Sheets and Steel Fiber Reinforced Concrete. Eng Struct 2018;176:812-24.

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Table 1. Test matrix

Beam	Wrapping scheme	Longitudinal BFRP layers	U-shape FRP layers	Rubber content (%)	Concrete strength (MPa)	U-shape fiber orientation
B0_2m	-	-	-	0	50.3	-
B0A_2m	A	4	-	0	50.3	-
B0B_2m	B	4	2	0	50.3	90^0
B0C_2m	C	4	2	0	50.3	90^0
B0D_2m	D	4	2	0	50.3	45^0
B15_2m	-	-	-	15	25.4	-
B15A_2m	A	4	-	15	25.4	-
B15B_2m	B	4	2	15	25.4	90^0
B30_2m	-	-	-	30	14.7	-
B30A_2m	A	4	-	30	14.7	-
B30B_2m	B	4	2	30	14.7	90^0

- Not applicable

Table 2. Experimental results of the impact tests

Rubber content (%)	Beam	Max Impact force (kN)	Impact force duration (ms)	Max reaction force (kN)	Max disp. (mm)	Residual disp. (mm)	Impact velocity (m/s)	*Residual velocity (m/s)	Imparted energy (J)	Dimensionless Imparted energy (J/kg)
0%	B0_2m	1475	0.69	54.8	45.6	32.9	6.25	-1.79	3744	
	B0A_2m	1815	0.54	51.7	33.4	16.1	6.23	-1.55	3801	
	B0A_2.5m	-	-	-	47.5	32.9	6.92	-1.54	4752	
	B0B_2m	1614	0.7	60.3	33.4	12.2	6.22	-1.55	3788	
	B0B_2.5m	1209	0.79	48.1	41.2	19.8	6.98	-1.55	4836	
15%	B15_2m	1412	0.58	45	43.4	29.8	6.19	-1.77	3673	
	B15A_2m	-	-	-	33.8	15.4	6.33	-2.11	3718	
	B15A_2.5m	1558	1.05	45.4	41.2	18.1	6.99	-2.10	4641	
	B15B_2m	-	-	-	31.4	10.7	6.30	-2.11	3679	
	B15B_2.5m	1173	0.98	54.4	43.1	25.7	7.24	-1.45	5253	
30%	B30_2m	-	-	-	44.8	31.3	5.99	-1.71	3441	
	B30A_2m	1552	0.65	39.8	36.6	-	6.29	-1.40	3926	
	B30A_2.5m	1351	1.00	42.9	46.9	19.5	7.04	-2.11	4709	
	B30B_2m	-	-	-	33.4	11.8	6.30	-2.19	3643	
	B30B_2.5m	1446	0.90	47.8	47.4	30.4	7.03	-1.41	4952	
0%	B0C_2m	1811	0.60	66.8	33.5	16.5	6.40	-2.40	3675	
	B0C_2.5m	1307	1.07	-	51.2	34.6	6.92	-0.77	4937	
	B0D_2m	1586	0.73	55.3	31.1	12.8	6.21	-2.33	3459	
	B0D_2.5m	1349	0.77	-	45.4	32.4	7.09	-1.57	4991	

- Not applicable * Negative sign indicates the travel direction upward