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Effect of Insufficient Tension Lap Splices on the Deformability and Crack Resistance of Reinforced Concrete Beams: A Comparative Study Techniques and Experimental Study

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ABSTRACT
In civil engineering, strengthening of reinforced concrete structural components is a challenging issue. This issue arises periodically across the world for a variety of causes, including incorrect design or construction, rehabilitation or strengthening for increasing service loads, and others. Additionally, preventing bond failure in the lap-splice zone also requires a sufficient bond between the concrete and reinforcement along the lap-splice length. Sometimes, the lap splice in the tension side has insufficient length for various reasons, such as design and construction errors. Accordingly, it is urgent that defective RC structures need strengthening to extend service life and reduce maintenance costs. This paper described the various materials and techniques that have been exploited for the rehabilitation/strengthening of existing RC structures with tension lap splices, including concrete or steel jacketing technique, fiber-reinforced polymer (FRP), external pre-stressing, near-surface mounted and fiber-reinforced composited materials. Moreover, the review aims to synthesize the data from papers and case studies around the world. This could serve as a useful reference for future studies and other researchers. A descriptive statistical analysis was performed on the collected data. Additionally, an experimental program involving five RC beams with tension lap splices of varying lengths (ranging from 10 to 60 times the rebar diameter) is conducted to investigate the effect of the lap splice length on the bending behavior of the beams. The results indicate that insufficient lap splice lengths adversely affect the behavior and failure mechanisms of the beams, leading to reduced ductility, flexural toughness, and bond strength. On the other hand, specimens with sufficient lap splice lengths exhibit improved deformability and crack resistance.
Keywords: *Strengthening, tension lap splice, reinforced concrete beams, deformability, crack resistance.*

1. Introduction

Rebar splicing is a crucial aspect of reinforced concrete construction [1, 2]. It is necessary when the length of reinforcing bars is insufficient to span the entire length of a structural element. Additionally, transportation constraints and site conditions can make handling and installing long reinforcement bars challenging, necessitating splicing techniques [3]. The lap splice is the most commonly economical and effective method for rebar splicing [4].

It involves overlapping two parallel rebars and securing them together using clamping devices or wire ties. Design codes or engineering standards typically specify the overlap length to ensure proper load transfer and bond between the rebars [5]. The lap splice can be classified into two main types: contacting splice and non-contacting splice [6]. In addition to the lap splice, there are other methods for rebar splicing. These include mechanical splices (using couplers, sleeves, or connectors), welded splices (where the rebars are welded together), and grouted splices (where the gap between the rebars is filled with grout) [3, 7].

Various parameters influence the performance of lap splices in reinforced concrete structures. These parameters include the lapped length (overlap length of the rebars), concrete cover (distance between the rebars and the outer surface of the concrete), lapped rebar percentage, reinforcement rebar diameter, consideration of transverse reinforcement near the splice, concrete mechanical properties, and the location of casting [8-9]. The structural performance of reinforced concrete structures depends critically on the bond behavior between the reinforcing steel and the surrounding concrete [10]. It affects the load transfer mechanism, crack development, and overall structural integrity. There are minimal inadequacies in load-carrying capacity and deformability under lateral loading when the lap splice length is applied in accordance with RC design guidelines [11, 12]. In situations when the retrofit might be more cost-effectively executed in phases, such as when reinforcing large spans or bridge beams that span several lanes of traffic, splicing becomes particularly useful [13]. Nevertheless, the structural element's strength and ductility are weakened when the lap splice length is inadequate [14]. Certain areas in reinforced concrete structures require special attention to bond behavior due to higher stress concentrations or complex loadings. Some examples of such regions include contra flexure points or continuous supports, deep beam supports, beam-column connections, and mid-span zones of simply supported beams [3, 7].

There has been extensive global utilization of reinforced concrete constructions. Much of the present-day structures have been constructed throughout the middle of the previous century. During that time, there was a marked shift in the need for designs, while the function of some structures was altered. Consequently, a number of structures fail to satisfy the criteria for serviceability, strength, and ductility set by contemporary standards such as the poor design of existing structures including reinforcement details at the splice zones [15, 16]. Additionally, lap splices can need strengthening because of inadequate bonding caused by a variety of factors, including mistakes in design and construction [17, 18]. The need for rehabilitation or strengthening of concrete structural components arises periodically across the world for various causes, including incorrect design or construction, rehabilitation or strengthening, increasing service loads, and others [19, 20]. A variety of materials and methods have been employed in recent years to strengthen and renovate existing reinforced concrete structures. Concrete or steel jacketing technique, externally bonded reinforcement (EBR) such as a fiber-reinforced polymer (FRP) and engineered cementitious composite (ECC), external pre-stressing, near-surface mounted, fiber-reinforced composited materials, and EBR on grooves (EBROG) are commonly available retrofitting schemes [21-23].

2. Strengthening Techniques

Indeed, the strengthening of inadequate lapped spliced elements in reinforced concrete (RC) structures can significantly reduce their susceptibility to earthquakes, thereby mitigating social and economic risks associated with such structures. While the improvement of structural elements can be complex and costly, it is a necessary step in construction. There are various reasons for the inadequate capacity of structural members, including outdated design codes, natural disasters, and environmental factors [17, 18]. These factors can weaken the structural elements, necessitating reinforcement to enhance their performance and safety. To improve defective structural elements, all available building materials must be utilized in the strengthening process. Before strengthening, a thorough evaluation and assessment of the deteriorated structural elements is essential to determine their current condition. This evaluation helps in estimating the strengthening requirements based on the in situ state of the structure. Different methods have been employed to enhance RC members with tight-lapped splices. These methods include spiral confinement, concrete jacketing with interior and exterior steel ties, jacketing using metal, fiber-reinforced concrete (FRC) and concrete jacketing, and external confinement using fiber-reinforced polymers (FRP) [23]. Each method offers unique advantages and can be selected based on factors such as the specific requirements of the structure, available resources, and desired performance objectives. By implementing these strengthening techniques, the load-carrying capacity, ductility, and seismic resistance of the inadequate lapped spliced elements can be significantly improved. This contributes to the overall safety and durability of the RC structures, reducing the risks associated with seismic events and ensuring their long-term functionality [3].

2.1 Externally Bonded Reinforcing Systems

In order to increase the flexural, shear, torsional, and axial sectional capacity of reinforced concrete structural components as well as to offer extra confinement and enhance the stability and serviceability of structural parts, externally bonded reinforcing systems, or EBRs, are employed [24]. Typically, two basic kinds of strengthening systems are used: one utilizes near-surface mounted (NSM) bars, and the other utilizes plates and/or sheets made of materials like carbon fibers (CFRP) or glass fibers (GFRP) [17]. Fixing strengthening sheets or plates to the tension side of the RC beam is the fundamental application of the EBR method for flexural strengthening. Comprehensive research was done on the failure mechanisms and behaviors of the RC beams strengthened by EB. The most prevalent and predictable failure mechanism is the soffit material plate's intermediate crack-induced debonding, also known as IC debonding [25]. Due to this failure pattern, the ultimate strength of strengthened beams is determined by regulations or standards in many countries [26]. When the IC debonding failure mode occurs, the strength of the bond between the EBR and the concrete determines the beam's maximum allowable strength [27], and the EBR is usually at a low-stress level at the ultimate state. To forecast the EBR-concrete interface's behavior, two models are suggested: bond-slip and bond-strength [28]. EB debonding not only induces early brittle failure but also restricts the enhancement of beam stiffness in the serviceability limit state. This is due to the fact that the stress activation occurs only after crack propagation on the concrete beam [29]. In accordance with the results of previous EBR investigations, Table 1 offers a comprehensive comparison. It is worth noting that the majority of prior research was carried out using embedded lengths that ranged from 5 to 40 times the diameter of the bar. These lengths were determined to be less than the standard bonded length as specified by national regulations/codes.

2.1.1 *Challenges of Employing Externally Bonded Systems (EBRs)*

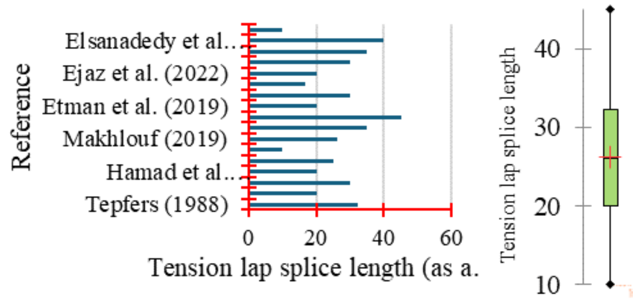
While previous research has highlighted the considerable benefits of employing externally bonded systems (EBRs) to strengthen concrete beams [30], their widespread adoption is impeded by several challenges associated with the lack of clarity regarding the durability of EBR-based repairs. Considerations about the durability of the composite-to-structure bond under harsh environmental conditions, including freeze-thaw cycles, high temperatures, and dry-wet conditions, are important when considering the use of EBR for civil structure repair [31]. To address the drawbacks of the EB method, other alternatives have been considered and investigated. Improving the bond quality is the first strategy. By using the near-surface mounted (NSM) method, strips or rods are affixed to the concrete surface via groove [32]. For design and analysis, bond strength and bond-slip models were also suggested [33]. To further improve the bond quality, anchors may be placed on the bonded interface to provide additional anchorages [34]. Furthermore, confinement serves to diminish the deterioration of the steel reinforcement's bond and restricts the extent of damage to the concrete inside the confined area. Consequently, this results in an enhanced capacity of the structure to absorb and dissipate energy [35].

2.2. Pre-Stress Technique

Employing the pre-stress technique is another crucial method for enhancing the effectiveness of strengthening [36]. In combination with EB or NSM approaches, the pre-stress system may be pre-tensioned, post-tensioned, or anchoring systems may be implemented concurrently [37]. Two fixed anchors are included in the strengthening system in addition to a tensioning mechanism. Applying epoxy glue to the bond surface comes before pre-stressing. In order to apply the tension stress, a hydraulic jack is then positioned between the fixed anchor and the response plate. Once the desired level of jacking force is achieved, the nuts inside the tensioning device are securely tightened. When the hydraulic jack is finally released, the plate's stress is controlled by the two fixed anchors. Despite ongoing approval, this approach needs further improvement [36]. Firstly, there is a restriction on design flexibility due to the size of the tensioning device. A short pre-stressed beam length could cause local shear failure if the tensioning mechanism is too close to the beam support. Secondly, the application of the strengthening technique requires epoxy adhesives, which generally have a curing period of at least 72 hours [37]. This curing time might lead to negative externalities, particularly when it comes to strengthening transportation infrastructures.

2.3 Substandard/insufficient bonded length

The term "substandard bonded length" refers to an embedded length that falls short of the minimum distance required by the code [12]. As to the Egyptian Code, the appropriate length for reinforcing bars under tension is 40-60 times d_b, while for compression it is 20-40 times d_b [3]. According to Table 1, which reviews previous research on the external strengthening of RC beams with tension lap splices using external methods, the range of insufficient lap length is 10 to 45 d_b, with a mean and median of 26.17 and 26.0 d_b, respectively. Table 2 presents descriptive statistics based on the previous review in Table 1. Figure 1 shows the values of tension lab splices and its statistics based on Table 1.



(a) tension lap splice lengths (b) Value distribuation

Figure 1- Values of tension lap splices and their statistics.

2.4 Summary of the Strengthening Methods

To conduct a comprehensive comparison of different strengthening methods for reinforced concrete (RC) members, it is necessary to conduct a comprehensive study using identical RC specimens, such as columns and beams, with the same testing setup. The only difference between the specimens should be the splice-strengthening approaches being investigated. In this section, a general comparison is made based on the findings from various studies discussed in the paper, as presented in Table 1. The findings from previous (Figure 2) studies indicate that UHPFRC at splice regions provides the highest increase in bond strength, while GFRP wrap offers the least improvement. Construction joints confined with steel stirrups and splices confined with FRP also enhance bond strength. However, it is crucial to consider various factors such as invasiveness, labor intensity, time consumption, disruption to building functionality, additional bulk, and cost when selecting the most suitable strengthening method for a specific project, especially in low- and middle-income developing countries.

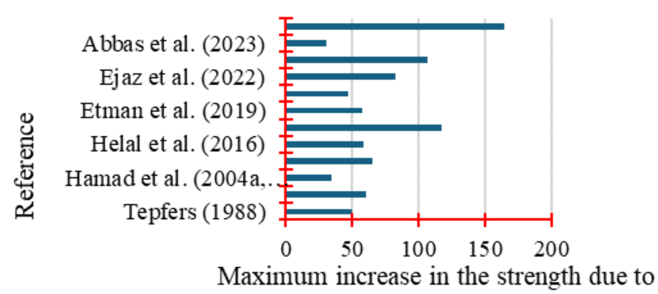


Figure 2- Earlier research's improvements in the bending strength (4-point flexure load) due to external strengthening.

Table 1- Summary of earlier investigations on external strengthening techniques of reinforced concrete elements with substandard lap splices.

Reference	Length of TLS (x d_b)	Method	ΔP (%)
[38]	32	Spiral confining	50 %
[39]	20	Confined with steel stirrups	50-60 %
[40-41]	20-30	GFRP wraps	8-34%
[42, 43]	20-25	Confined with FRP layers	30-70 %
[44]	20	FRP and textile reinforced mesh	35 %
[45, 46]	25	Externally confined	14-65%
[47]	10	Post-tension method utilizing strap	58 %
[17]	26-45	CFRP, and GFRP strip layers	9.70-117 %
[48]	20- 55	Jacketing of UHP-SHCC	6-57 %
[49]	5	Internal and external confinement by CFRP sheets	40 %
[18]	16.67	NSM and NSM–CFRP	19.95-46.76%
[50]	20-35	Hollow steel section	10.3-82.6 %
[51]	40	Externally attached CFRP	106%
[52]	65	Full-scale glass and FRP reinforced beam	2-30 %
[21]	10	ECC ferrocement layers	114-164%

GFRP: Glass Fibre Reinforced Concrete; CFRP: Carbon Fiber Reinforced Polymer; SFRC: Steel Fiber Reinforced Concrete; Ultra-High-Performance Strain Hardening Cementitious Composites; TLS: tension lap splice. ΔP is the increase ratio in the strength due to strengthening.

Table 2- Descriptive Statistics of Insufficient Tension Lap Splices.

Obsers.	Min.	Max.	1st Quartile	Median	3rd Quartile	Mean	Standard deviation
17	10.0	45.0	20.0	26.0	32.2	26.169	9.839

Generally, tension lap splices are critical components in reinforced concrete structures as they transfer load between adjacent reinforcement bars. Insufficient lap splice lengths can compromise the structural integrity of the beams, leading to potential failure and safety hazards. Understanding the effects of such deficiencies is crucial for ensuring the safety and reliability of concrete structures. Also, deformability and crack resistance are essential properties of reinforced concrete beams. These characteristics determine the ability of structures to withstand applied loads, resist cracking, and maintain their functionality over the design service life. Investigating the influence of insufficient tension lap splices on these parameters provides valuable insights into the performance limitations and potential vulnerabilities of concrete beams. Accordingly, an experimental program involving five RC beams with tension lap splices of varying lengths (ranging from 10 to 60 times the rebar diameter) is conducted to investigate the effect of the lap splice length on the bending behavior of the beams.

3. Experimental testing program

3.1 Materials

3.1.1 Concrete

Normal concrete (NC) has been employed to cast five RC beams. NC was composed, as stated in Table 3, of the following: sand as the fine aggregate, graded crushed basalt dolomite with a maximum nominal size of 15 mm as the coarse aggregate, 42.5 grade Portland cement, water, and superplasticizer. The compressive strength of the NC mix was set at 30 MPa. Three cubes (150x150x150 mm^3) were taken from the mix to specify its compressive strength. As well as three cylinders (150x300 mm) were also taken to specify its tensile splitting strength.

Table 3- The Mix Proportions of Normal Concrete.

Water	Cement (C)	Sand	Basalt dolomit	Superplasticizer	Water–Cement ratio (W/C)
kg/m^3					-
150	300	650	1290	2	0.50

3.1.2 Reinforcing steel

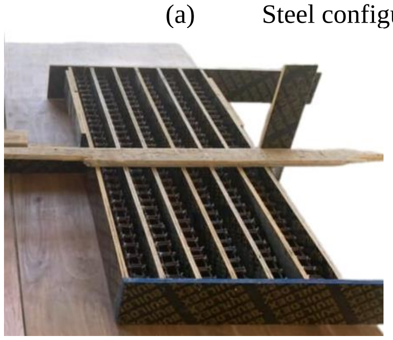
In this study, normal mild steel (NMS) and high tensile steel (HTS) were used as internal steel rebars. NMS was a circular bar with a smooth surface and 8.0 mm diameter. Meanwhile, HTS was a circular bar with ribs on its surface and was 10.0 mm in diameter. For casted beams, NMS was used as secondary reinforcement as well as transversal reinforcement (stirrups) whilst HTS was employed as primary reinforcement. To ascertain the employed steel bars' mechanical characteristics, tension tests were carried out.

3.2 Specimen preparation

The test specimens consist of five beams distributed as follows: one specimen without tension lap splice, one four specimens with tension lap equal 60$d_{b,}$ 40d_b, 25d_bm 10$d_{b,}$ respectively. Table 4 summarizes the details of the specimens. According to that, all the studied specimens had the same geometrical dimensions, with 100 mm width, 200 mm depth, and a span of 1500 mm. All the specimens have a 15 mm concrete cover. Two 10 mm-diameter deformed HTS bars were used as bottom reinforcement, whereas two 8.0 mm-diameter smooth NMS bars were used as top reinforcement. Stirrups with a diameter of 8.0 mm were arranged at a center-to-center spacing of 50 mm within the total length of the beam span to prevent shear failure [53].

The specimens were cast by pouring the concrete mixture layer by layer along the vertical axis. This process helps in achieving uniform compaction and eliminates potential voids or air pockets within the concrete. After 24 hours, when the concrete had gained sufficient strength, the specimens were carefully removed from the mold. The de-molded specimens were then transferred to a curing room with a controlled environment. This curing room maintained a temperature of 20℃±2℃ and a relative humidity of 95% RH. Following the initial curing period, the specimens were stored in a fog room. The fog room is maintained at a temperature of 23±2℃. This additional storage period allows for further hydration and the development of strength in the concrete before testing. Figure 3 indicates images from specimen perpetration.

(a) Steel configuration of tension lab spliced specimens.



(b) Reinforcements used within the forms. (c) After pouring.

Figure 3- Specimen Perpetration.

Table 4- Details of Specimens and Strengthening Parameters.

Number	Specimen ID	Specimen dimensions	Steel reinforcement			
			bottom	top	stirrups	Lap splice
		mm				mm
1	B_0	100x200/1500	2φ10	2φ8	φ8@ 50 mm	-
2	B_{L10}					$10d_b$
3	B_{L25}					$25d_b$
4	B_{L35}					$35d_b$
5	B_{L60}					$60d_b$

Where d_b is the main steel diameter of bar reinforcement.

3.3 Testing set-up and Measurements

3.3.1 Compressive strength test

In this study, the 28-day compressive strength of the concrete specimens was determined using 150 x 150 x 150 mm³ cube samples, with an average strength of 30.5 MPa. This value represents the average strength achieved by the concrete after the 28-day curing period.

3.3.2 Tensile strength test

The 28-day tensile splitting strengths were examined using 150 x 300 mm sample cylinders, and the average tensile strength of three 150 mm cylinders was 2.74 MPa.

3.3.3 Four-point loading test

A laptop was paired to a UCAM-550A strain data logger device, which automatically captured all test data. To conduct the flexure test, the specimen was placed on a hinge at one end and a rolling support at the other. The force-controlled vertical application of the load was performed using a load cell. Two smooth steel rollers, one-third, and two-thirds of the span apart from one end, transferred the vertical load P from the central span to the specimen (Figure 4). A linear variable differential transducer (LVDT) was employed to measure the mid-span deflection of the specimen. Figure 4 depicts the experimental test set-up. The loading rate of 0.5 kN/s was used. The first crack, failure mode, and maximum failure load were documented.

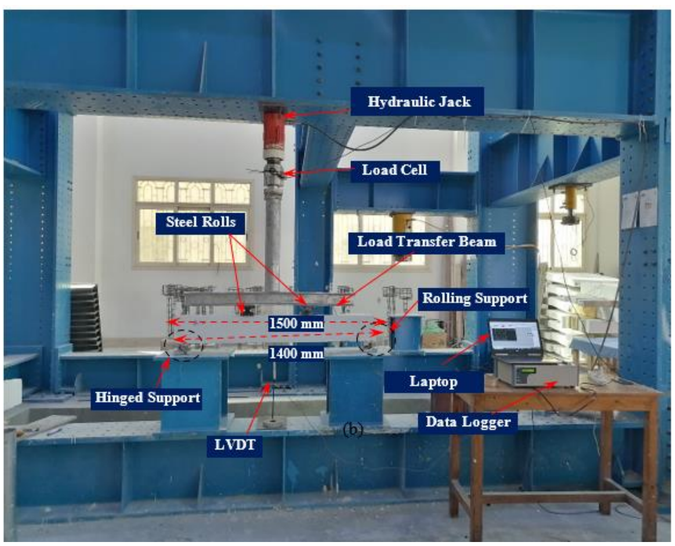


Figure4- Four-point loading test set-up.

4. Test results and analysis

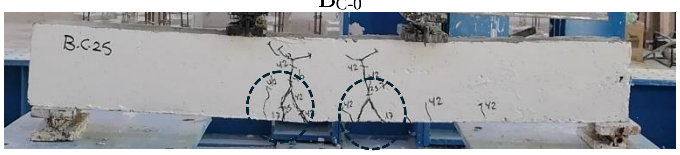
4.1 Failure modes

Figure 5 illustrates observations on the crack's formation and the failure mechanism of the tested specimens under a four-point loading test. The specimen B_0, which did not have a tension lap splice, exhibited a particular behavior during the experimental testing. According to Figure5a, as the load on the specimen gradually increased towards its ultimate value, vertical crack damage started to propagate at the midspan in the tension zone. These cracks then spread out between the two points of loading. This behavior is commonly observed in reinforced concrete beams subjected to bending forces. At the ultimate load, as expected, the failure pattern of the B_{C-0} specimen was characterized as a typical flexure failure pattern in the pure bending region.

For the specimens with the substandard tension lap-splice region ($B_{L10,}$ $B_{L25,}$ and B_{L35}), flexure cracks initially appeared at the ends of the lap zone. These cracks then widened, but their number did not increase significantly. The failure was characterized by a limited number of wide flexural cracks in the concrete near the end of the lap, indicating slippage of the tension rebars, as seen in Figure 5. This could be explained by the fact that the lap splice length or the embedment length of the rebar is insufficient; accordingly, it led to inadequate bond strength between the rebar and the concrete, resulting in slippage. This slippage can cause a localized stress concentration at the ends of the lap zone, leading to the formation and widening of flexure cracks. The limited number of wide flexural cracks observed in the concrete near the end of the lap zone indicates the occurrence of such slippage. The behavior of cracking for specimen B_{L25} was somewhat similar to the behavior of specimen B_{L35}, however, specimen B_{L35} was able to withstand higher bending loads. In addition, specimen B_{L10} showed sudden collapse due to bending loads (Figure 5c)

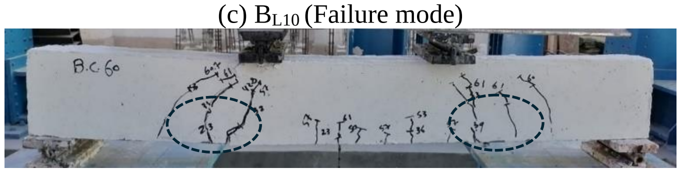
Flexure cracks initially appeared at the ends of the lap zone in the B_{L60} specimen (Figure 5d). These cracks then propagated towards the tension zone (mid-span) and increased in size and number as the load was increased. In this case, the length of the lap splice did not have a negative effect on the behavior of the specimen; instead, it improved the behavior because the length of the lap splice was sufficient. An extensive number of wide flexural cracks in the concrete near the end of the lap characterized the failure of the BC-L60 specimen. This pattern of cracks further confirms the flexural failure mode. In summary, the key difference in behavior between the specimen with a sufficient lap splice length (B_{C-L60}) and the one with an insufficient lap splice length (B_{C-L25}) lies in the effectiveness of the bond between the rebars and the concrete. A sufficient lap splice length ensures a stronger bond, leading to improved load transfer and more desirable flexural failure behavior. Conversely, an insufficient lap splice length compromises the bond, resulting in a different failure mode and potentially premature failure.

B_{C-0}



(b) B_{L25}

(c) B_{L10} (Failure mode)



(d) B_{L60}

Figure 5- Failure patterns and cracks of tested specimens.

4.2. load-deflection relations

The experimental evidence in this section focuses on cracking behaviors and load-deflection relationships of the tested specimens. The load-deflection analysis yielded several important findings, which are summarized in Tables 6 and 7.

Furthermore, Figure 6 displays the load-deflection curves for the tested beams. These curves visually represent the relationship between the applied vertical load and the corresponding mid-span deflection for each group. These curves typically exhibit three distinct stages: elastic, elastic-plastic, and softening. In the elastic stage, the load increases linearly with deflection. This stage indicates that the specimen's behavior is initially elastic, meaning it can resist the applied load without significant deformation or damage. As the displacement continues to increase, the load starts to increase nonlinearly, marking the elastic-plastic stage. During this stage, the specimen undergoes deformation and experiences some degree of permanent deformation, indicating a transition from elastic to plastic behavior. The load-deflection curve rises until it reaches the peak load, representing the specimen's maximum capacity to resist the applied load. After reaching the peak load, the load gradually decreases as deflection increases. This stage is known as the softening stage, where the specimen experiences a reduction in load-carrying capacity and may exhibit a decrease in stiffness. The softening behavior is typically associated with the development of cracks, damage, or failure mechanisms within the specimen. The post-peak response, ultimate load, and deflection response were enhanced due to the increase in the tension lab splice length. The enhancement in the load-deflection curves proves that the tension lab splice length significantly affects the ductility and load-carrying capacity of the specimens.

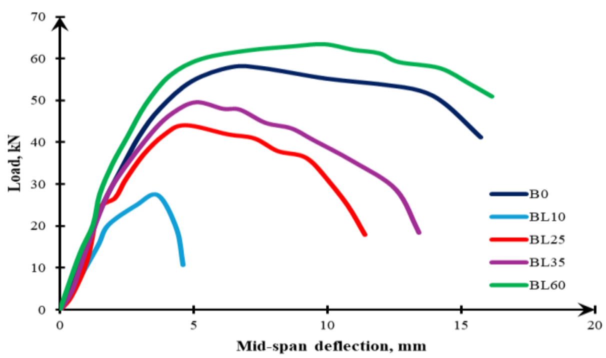


Figure 6- Load–Deflection Curves (mid-span deflection for all specimens).

4.2.1. First crack and ultimate loads

The results presented in Table 6 indicate the performance of the tested specimens at the critical and ultimate stages. Regarding the first crack appearance, specimen B_{L10} exhibited weak resistance compared to the control specimen (B_0). The rate of first crack appearance in B_{L10} was 43.28% lower than that of the control specimen. This suggests that the shorter tension lap length in B_{L10} led to a reduced ability to resist cracking under applied loads. On the other hand, specimen B_{L60} showed a higher resistance to the appearance of the first crack compared to the control specimen. The percentage of first crack appearance in B_{L60} was 7.45% higher than that of the control specimen. This indicates that the longer tension lap length in B_{L60} contributed to a better ability to withstand cracking. Overall, these results demonstrate that the tension lap length plays a significant role in the resistance to cracks. Longer tension lap lengths tend to improve the specimen's ability to resist the initiation of cracks. The correlation between tension lap length and crack resistance suggests that proper detailing and design considerations regarding lap length can enhance the structural performance and durability of reinforced concrete elements. The same trend was observed for the ultimate capacities of the tested specimens. The range of percentage change values in ultimate load varied from a high of 5.86% for specimen B_{L60} to a minimum of -53.45% for specimen B_{L10} relative to the values of the control specimen (B_0).

Table 5- Details of Critical and Ultimate Stages.

Group ID	Specimen ID	Cracking Stage	Ultimate Stage	
		Increase in P_c	Increase in P_{ult}	Δ_p
		%	%	Mm
G_I	B_0	-	-	7.64
	B_{L10}	-43.28	-53.45	3.47
	B_{L25}	-18.28	-27.55	4.66
	B_{L35}	-12.45	-15.50	5.07
	B_{L60}	7.45	5.86	9.12

4.2.2. Ultimate deflection and ductility

The peak deflection (Δ_p) is the deformation corresponding to the peak/ultimate load. Brittle failure caused by sudden fracture of the specimen should be avoided to assure large ultimate deformation (high enough ductility). According to Table 6, the Δ_p for the lap control specimen, B_0 was 7.64 mm. The average values of the reduction in Δ_p were recorded as 54.39%, 39 and 33.63% for specimens B_{L10}, B_{L25}, and B_{L35}, respectively, with a mean value of 41.21% compared to specimen B_0. While specimen B_{L60} showed an improvement in the Δ_p with 19.37% compared to the control specimen.

Table 6- Details of Deformability, Ductility Ratios, and Toughness.

Group ID	**Specimen ID**	Ultimate deflection	Flexure toughness	Ductility
		Increase in Δ_p	$\Delta\Psi$	$\Delta\mu_p$
		%	%	%
G_I	B_0	-	-	-
	B_{L10}	-54.58	-85.41	-46.22
	B_{L25}	-39.0	-51.38	-28.44
	B_{L35}	-33.63	-31.25	-22.66
	B_{L60}	19.37	25.69	22.67

The purpose of the ductility analysis is to assess the effect of the tension lap splice on the ductility of the strengthened beams. The findings of this analysis are displayed in Table 6. The peak deflection ductility index (μ_p) is determined by dividing the peak deflection (Δ_p) by the yield deflection (Δ_y) [54]. Generally, the insufficient tension lap splice in specimens B_{L10}, B_{L25}, and B_{L35} lead to a significant reduction in the ductility index with a rate of 46.22, 28.44, and 22.66%, respectively. While for sufficient tension lap splice in specimen B_{L60}, the specimen showed an improvement in the ductility index with a rate of 22.67% compared to the control specimen.

4.2.4. Flexural toughness (ψ)

As an indication of the specimens' capacity to withstand the formation of cracks, toughness has been proposed as a metric for assessing deformability (or fast fracture) [55]. In this study, flexural toughness (Ψ) is defined as the area under the load-deflection curve. Notably, all curves were finished at 0.8 f_{bu} after the peak. As presented in Table 5, the flexural toughness (Ψ) of the control specimen B_0 was measured to be 720 kN.mm. However, for the insufficient tension lap splice in specimens B_{L10}, B_{L25}, and B_{L35}, there was a notable reduction in flexural toughness (ΔΨ). The reduction in flexural toughness ranged between 31% and 85%, with an average increase of 55.68% compared to the control specimen B_0. These findings suggest that the insufficient tension lap splice adversely affects the deformability and crack resistance of the specimens. The specimen (B_{L60}) with sufficient lap exhibited significantly higher flexural toughness values, indicating their improved ability to withstand crack growth and resist fast fracture compared to the control specimen.

5. Conclusions

This paper described the various materials and techniques that have been exploited for the rehabilitation/strengthening of existing RC structures with tension lap splices, including concrete or steel jacketing technique, fiber-reinforced polymer (FRP), external pre-stressing, near-surface mounted and fiber-reinforced composited materials. Moreover, the review aims to synthesize the data from papers and case studies around the world. This could serve as a useful reference for future studies and other researchers. A descriptive statistical analysis was performed on the collected data. Additionally, an experimental program involving five RC beams with insufficient tension lap splices of varying lengths (ranging from 10 to 60 times the rebar diameter) is conducted to investigate the effect of the lap splice length on the bending behavior of the beams.

1. Based on the descriptive statistics, the range of insufficient lap length is 10 to 45 d_b, with a mean and median of 26.17 and 26.0 d_b, respectively.

2. In conclusion, the results of previous studies indicate that Ultra-High-Performance Fiber-Reinforced Concrete (UHPFRC) at splice regions shows the highest increase in bond strength, while Glass Fiber-Reinforced Polymer (GFRP) wrap provides the least improvement.

3. Construction joints confined with steel stirrups and splices confined with FRP also enhance bond strength. However, when selecting a suitable strengthening method, it is essential to consider factors such as invasiveness, labor intensity, time consumption, disruption to building functionality, additional bulk, and cost. These considerations are especially important in low- and middle-income developing countries.

4. Based on the experimental results, insufficient lap splice length resulted in slippage of tension rebars and limited, wide flexural cracks near the end of the lap zone.

5. the presence and length of the tension lap splice significantly affect the behavior and failure mechanisms of reinforced concrete beams. Specimens without a splice showed typical flexure failure. Substandard lap splice lengths resulted in limited and wide flexural cracks due to rebars slipping.

6. Specimens with sufficient lap splice lengths exhibited desirable flexural failure behavior with extensive flexural cracks.

7. Specimen B_{L60} showed the highest percentage change in ultimate load (+5.86%) compared to the control specimen, and specimen B_{L10} showed the lowest percentage change (-53.45%). The same trend was observed for the first crack load capacities.

8. Specimens with insufficient lap splice lengths (B_{L10}, B_{L25}, and B_{L35}) experienced significant reductions in the ductility index, with average rates of 46.22%, 28.44%, and 22.66%, respectively, compared to B_0. While specimen B_{L60}, with a sufficient lap splice length, demonstrated an improvement in the ductility index of 22.67% compared to B_0.

9. Specimens with insufficient tension lap splices showed a notable reduction in flexural toughness (ΔΨ). The reduction ranged from 31% to 85%, with an average increase of 55.68% compared to the control specimen. On the other hand, the specimen with a sufficient lap splice (BL60) exhibited significantly higher flexural toughness values, suggesting the improved ability to withstand crack growth and resist fast fracture compared to the control specimen.

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