Response of Reinforced Concrete Dapped-End Beams Exhibiting Bond Deterioration Subjected to Static and Cyclic Loading

Ajibola Ibrahim Quadri¹* and Chikako Fujiyama²

Received 3 March 2021, accepted 2 May 2021 doi:10.3151/jact.19.536

Abstract
In existing bridge structures, reinforced concrete dapped-end beams/girders (RCDEB) are frequently subjected to service loadings that exceed their design capacity, due to increasing economic and population growth. Dapped-end beams are prone to the accumulation of water due to improper drainage and sealing of the joint, providing favorable conditions for corrosion due to the stagnation of chloride rich water from de-icing salts used on roads. The situation can get even worse when freezing and thawing due to extreme weather conditions is present. Due to difficulties in maintenance of the dapped-end regions, this often leads to the deterioration of the concrete around the recess and bond deterioration, which ultimately results in durability issues. Unfortunately, the performance of RCDEB exhibiting corrosion induced bond deterioration is still not well understood. Hence, experimental, and numerical investigations were conducted to understand better the response of RCDEB subjected to static and cyclic loading, with the presence of bond deterioration. Furthermore, the static capacity of the beam was adopted to consider the variable amplitude cyclic loading at an increment of 15% static capacity for every 20 cycles. The reliability of the numerical examination under the direct-path constitutive models of concrete was extended to the moving load scenario by applying a lower load amplitude than the static capacity. It is shown that the prominent failure mechanisms observed in both static and cyclic tests were diagonal tension and shear. Through numerical investigations, it is also shown that from the damage level developed in the RCDEB under the moving load at a relatively low magnitude, high stresses were found within a relatively short number of cycles, which raises a serious cause for concern.

1. Introduction
Reinforced concrete Dapped End Beams (RCDEB) are employed in bridges and many other pre-cast constructions to reduce end depth and increase lateral stability due to the notching or recessing at the ends. Besides, RCDEBs allows a reduction in the total height of constructions. They experience recurrent loads when used in bridges, the cross-sections of RCDEBs utilized in construction is weakened at the reentrant corner close to the support because of the discontinuity region (D-region) (Mattock and Chan 1979), this disturbs the flow of internal stresses in the members and causes a stress concentration at the reentrant portion of the beam, which can create crack development if enough stiffness is not provided (Moreno-Martinez and Meli 2014). Furthermore, investigating the anchorage zone of RCDEB in the bridge girder is often difficult due to the recess portions that sit on other structural members, water or eroded materials can accumulate on this part, which accelerates the deterioration of the beam leading to corrosion concern. A case of the collapse of the De la Concorde Bridge in Montreal, Canada in 2006 (Fig. 1) due to corrosion of the reinforcement at the critical portion has been identified (Johnson 2007). Durability is a constant threat for the DEBs when exposed to recurrent loading from traffic volumes (Gjørv 2011). Moreover, the spikes in economic activities over time also subject some of the bridges to overloading more than their designed capacity.

A series of examinations have been carried out to explore the shear capacity of RCDEBs. The first investigation conducted on the dapped end beam was made by (Reynolds 1969) who made a proposal on the reinforcement detailing and established measures on the equilibrium of the bending moments for appraising the strength contribution of each type of reinforcement. Hoogenboom et al. (2005) have examined the shear strength of dapped-end beam (DEB) and recommended that the depth of nib should be more than 0.45 times the full depth of the beam and suggested that the spacing of the first stirrup from the end face of the dapped end beam must be less than 40 mm. Mohammed et al. (2019) studied the performance of RC and R-ECC DEB based on the role of the hanger and the diagonal reinforcement, although the experiment was proved to be appropriate, an increase in capacity was observed with a certain extent of delay to crack initiation, the undapped zone lost its confining strength which drastically resulted in damage at the bottom. Lu et al. (2015) examined 24 RCDEBs with shear span to depth ratio greater than unity and concluded that the shear strength of DEBs increased con-
siderably with an increase in the concrete compressive strength, and with a smaller shear span to depth ratio of the DEB, stiffness and ultimate load increased considerably. Syed et al. (2019) conducted a parametric study under dynamic loading by adopting different reinforcement detailing. Under the impact loading, the shear strength of DEBs increases as concrete compressive strength increases. Shakir and Abd (2020) considered shear slenderness ratio of 1 and 1.5 for a half joint beams strengthened with different arrangement of FRP sheets, the result showed that when a reduced shear span of half joint beam is used, the stiffness and ductility is enhanced.

There is, however, a paucity of an experimental investigation on the response of the RCDEBs subjected to fatigue loading conditions under the influence of the bond deterioration between the reinforced concrete composite of RCDEBs. Moreover, several RC structures experience cracks and defects from fatigue degradation due to constant overloading/traffic and environmental concern that reduces the life expectancy of the structures. Thus, investigation of the RC bridges subjected to fatigue problems gives much concern to the engineers these days (Maekawa and Wang 2020). Scholars have conducted in-depth studies on the impact of fatigue on the overall mechanical performance of RC structures. Gebreyouhannes et al. (2008) experimentally investigated the fatigue shear response of RC beams subjected to fixed pulsating and moving loads in form of moving vehicles in traffic states using strain and time-dependent fatigue constitutive models in multi-scale. The moving load was found to cause a drastic reduction in the fatigue life of RC beams due to the aggregated stress reversal and variation in shear force to the static capacity of the beam. Fathalla et al. (2018) presented a life assessment of concrete bridge deck by deriving the hazard cracks pattern through an artificial neural network, the locations and widths of concrete cracking are converted to space-averaged strains in space for predicting future occurrence. The fatigue reliability model of orthotropic bridge slabs was conducted by Farreras-Alcover et al. (2017) by adopting ambient temperature, strain monitoring, traffic, and made stress-cycles curves applicable to strain monitor. Hiratsuka et al. (2016) reported a structural behavior of RC bridge slabs subjected to different loading histories through the path-dependent high cycle fatigue loading in the laboratory; the same scenario was simulated using a nonlinear FE analysis approach. Approximately the same behavior was observed. This is useful in practice when considering the gradual damaging of the concrete structures under high cycle fatigue loading. Fujiyama et al. (2013) presented a numerical scheme to evaluate a residual fatigue damage life of RC bridge decks using a pseudo-cracking approach that converts cracks data investigated on-site to the real-life scenario. Here, the past loading history of several inspected bridges subjected to real traffic loads was considered, the quantitative results show approximately good agreement with the specifications of Japanese maintenance codes. Some of these approaches are adopted for engineering cracks verification on RC bridge decks. Fatigue failures of reinforced concrete beams under shear and flexure have also been examined (Suryanto et al. 2019; El-Kashif and Maekawa 2004). Fatigue investigation of RCDEB is however not common in literature. Moreno-Martínez and Meli (2014) investigated a precast pre-stressed dapped-end girder similar to those used in an elevated viaduct in Mexico City; premature cracking was recorded in the reentrant corner of the beam under service loading. It is therefore essential to comprehend the impact of fatigue loading on RCDEB especially the consideration of moving type load as it can potentially reduce the life of concrete structures in service.

1.1 Research significance
From the reassessment of the existing literature, it is set up that numerous assorted works were led on the response of RCDEB under static loading conditions. The

Fig. 1 The condition of the De la Concorde Gerber Bridge girder in Canada due to corrosion prior to collapse (Johnson 2007).
accompanying issues are yet open and should be tended to.
1. Experimental and analytical examinations of fatigue behavior of RCDEB are insufficient.
2. The influence of bond deterioration of the RC composite, at the anchorage zone of the RCDEB, on the load-carrying capacity and the influence of such deterioration under static and fatigue loading conditions is not clearly understood.

Considering these, in the current investigation, experimental and numerical examinations are conducted to comprehend the behavior of RCDEB under bond deterioration issue subjected to monotonic and fatigue problem in form of cyclic loading.

1.2 Bond deterioration concern

When a deformed bar is used as tensile reinforcement, the rebar lugs create the bond with the surrounding concrete, the presence of cracks in concrete around the reinforcement are not only related to durability but also characterized the bond mechanism between the reinforcement and the concrete. Thus, Understanding of bond effect in RC is accordingly critical (Goto 1971). Furthermore, the performance of cracks of concrete in facilitating the corrosion of reinforcement in RC structures remains questionable. On one hand, cracks reduce the life expectancy of the structures because they permit carbonation, thus allowing chloride ions, moisture, and oxygen ingress towards the reinforcement. On the other hand, even though cracks hasten the occurrence of corrosion, their actions are confined to the bar length parallel to the width of the crack. Hence, concrete quality and cover thickness are important parameters for preventing corrosion of steel in concrete. Eurocode 2 (EN-2 2005) recommends a semi-empirical approach to assess the crack width in terms of the strain difference between the concrete and the reinforcement and maximum crack spacing, among different other parameters. The long-term impact of creep could have a significant drawback on the performance under recurrent loading and explicitly on the size and width of cracks.

1.3 Design criteria for the dapped-end beams (DEBs)
The detailing of the reinforcement requirement of the DEBs was done in accordance with the specification of Eurocode 2 (EN-2 2005), which embraces the “struts-and-tie” inclination approach for the design of shear in structures with discontinuity sections where a non-linear strain distribution exists such as deep beam, corbel, and DEB, which results in the economy of the amount of shear reinforcement required. The concrete stress emanating from the nib is resisted by the tension force in the web reinforcement located near the full-depth end face of the beam (Fig. 2a). In addition, the strength of the undapped end of the DEB is also influenced by the inclined tension crack that is primarily initiated at the recessed corner and from the full depth corner of the beam. Adequate reinforcement must be provided to resist the crack to forestall failure. Cook and Mitchell (1988) developed a strut-and-tie model that was later adopted in the ACI building code requirements, ACI 318-95 (ACI 1995) and the new revision, ACI 318-19 (ACI 2019), as presented in Fig. 2b. The model shows a compression strut engaging an inclined crack, which does not depict the recommendation for the present study. Another option is presented in CEB-FIP recommendations for practical design of structural concrete (fib 1996) as shown in Fig. 2c, which is similar to what is adopted in the present study. However, modifications by Arya (2009) and by Mari et al. (2015) were adopted.

2. Experimental study

To evaluate the behavior of the DEB under the bond deterioration, six dapped end beams have been designed and cast. 9 mm diameter deformed bars are used to resist the forces originated from the nib region at 150 mm spacing, while 210 mm spacing is adopted for the stirrups at the middle portion of the beam. 22 mm deformed bars are used for all the longitudinal reinforcement both at the reentrant corner (nib) and at the full bottom depth of the beam as presented in Fig. 3, so that the reinforcements can attain their yield strength before the ultimate failure. The yield strength and the ultimate strength of the 9 mm rebar used are 499 MPa and 614 MPa respectively, while that of 22 mm bar (D22) used for each beam is shown in Table 1. PVC hollow pipe of 140 mm long (equivalent to 6.4D where D is the diameter of 22 mm bar inserted in the hollow pipe) and 27 mm inner diameter was used to create the bond deterioration between the longitudinal reinforcement and the concrete. Silicon gum was infused inside the PVC hollow pipe, and the longitudinal reinforcement was inserted into the PVC pipe as depicted in

![Fig. 2 Struts-and-tie approach for DEB: (a) for this study, (b) design recommendation in ACI 318-95, (c) FIP recommendation.](image-url)
The total span of the RCDEB is 1600 mm, the distance between the supports is 1200 mm, the dimensions are 300 mm long, 200 mm high, and 200 mm wide. The undapped section has the dimension of 1000×200×400 mm length, width, and height respectively. 6 beams labeled (S0, S1, … S5) were examined under static and fatigue problem in terms of cyclic loading. The appearance of the beams is shown in Fig. 3. S0 is a control beam without the bond loss loaded monotonically at shear span to depth ratio \((a/d=1\), where \(d\) is the total depth of the undapped section). The position was adopted to capture the diagonal cracks from the full depth of the beam. S1 and S5 have the same bond loss at the nib section, while the bond loss in S3 and S4 is included at the flexure zone. S5 was also subjected to static loading at the midpoint which equivalent to a/d>1, while the rest of the beams were tested under the cyclic conditions. S2 and S3 were loaded at \(a/d=1\), and S1 and S4 were loaded cyclically at the midpoint. The arrangement of the bond loss for each beam and description is given in Fig. 3 and Table 1.

M35 grade of a concrete mix has been designed to accomplish 28-day characteristic cylinder compressive strength of 35 MPa. Constituents of the concrete mix are ordinary Portland cement, fine aggregate, and coarse aggregate with a maximum aggregate size of 20 mm mixed at a ratio of 1:1.6:2.94 by weight of cement. A water-cement ratio (w/c) of 0.43 and air-entraining water-reducing agent ranges between 0.3 and 0.5% by weight of cement are adopted. In addition to the test beams, cylindrical sizes of 100 mm diameter and 200 mm height were cast for each beam to determine the compressive strength and split tensile strength of concrete. The slump and air entrainment values achieved during testing were 110 mm and 1.75%, respectively. From the test material, the results of the compressive and the split tensile strength for each beam are presented in Table 1. The same values were incorporated in the numerical simulation studies. The splitting tensile strength (\(f_{ct}^{s}\)) is computed using Eq. (1). The loading test was carried out after curing the beam for 28 days.

\[
f_{ct} = \frac{2P}{\pi ld}
\]

where \(P\) is breaking load, \(l\) is length of cylinder, \(d\) is diameter of cylinder.

![Fig. 3 Reinforced concrete dapped end and bond loss detailing.](image)
2.1 Instrumentation

The specimens were instrumented to obtain as much information as necessary, most importantly at the dapped section where the bond loss was made. All the results were recorded automatically with the aid of a data logger. Steel strains were measured using 5 mm electrical resistance strain gauges attached to the longitudinal reinforcement inside and outside of the PVC hollow pipe in the nib and at the flexure zone. The strain gauges were also glued to the hanger reinforcement on the North and South sides of the beam to evaluate the strain values develop in the reinforcement during loading. To capture the vertical displacement of the beam, three 25 mm capacity linear variable differential transducers (LVDTs) were positioned on the beam, at the lower portion of the loading point, and the upper side of the supports. The test beams were subjected to three-point-bending in a universal testing machine with a total capacity of 2000 kN (Fig. 4). A top steel rigid plate 200×200 mm was used to transfer the load from the machine head to the beam. The load was applied at an increment of 5 kN until failure in the case of static loading. The static capacity here was adopted for determining the fatigue cyclic loading criteria. In the case of the cyclic loading, a minimum of 15% static capacity was adopted with an increment of 15% capacity loading at every 20 cycles until failure. The minimum load amplitude was maintained at 5% static capacity to ensure contact between the loading device and the test specimen. The cyclic amplitude loading was done in the form of increasing steps as shown in Fig. 5.

3. Fatigue constitutive model for concrete

The constitutive model adopted in this investigation is briefly summarized in Fig. 6. Starting from the mechanical response of cracked reinforced concrete skeleton considered based on the time-dependent nonlinear constitutive law (Maekawa et al. 2006b). The reinforced concrete stress is a simple summation of the cracked reinforced concrete model and the embedded reinforcement. To develop the stress-strain relationship of the cracked concrete, the combination of the tension softening-stiffening model normal to cracks, compression-tension model along the cracks, and the shear transfer along the cracks are imperative. In a scenario of moving wheel load such as the traffic conditions, the multi-directional fixed crack approach is considered on the grounds that the reinforced concrete structure sustains cracks in different directions. Maekawa et al. (2006a) incorporated the dynamic response of the reinforced concrete structures by considering the direct integral path-dependency of the multi-directional fixed cracks. The one-dimensional stress-strain relation is built up in 3-dimensional space average constitutive law. When there are several cracks, the active crack is considered to control the whole nonlinearity response of the reinforced concrete structure, which is identified to have the largest crack width among the cracks. Finally, the high fatigue constitutive model of concrete in compression, tension, and shear transfer across the reinforced concrete (RC) presented by (Maekawa et al. 2006c), have been introduced into the time-dependent constitutive model of concrete structures. The governing equations have been presented by (Maekawa et al. 2006c).

3.1 Numerical study

Experimental examination and computational simulation of reinforced concrete structures are frequently adopted to analyze the nonlinear behavior of concrete structures. While the former gives accurate results, it is restricted to
the understanding of occurrence under limited geometries, loading, boundary conditions, and cost. The latter entails unrestricted computer modeling of reinforced concrete. The numerical analysis is thus adopted as an alternative to the rigorous and dear experimental study. The numerical study aims to consider the applicability of the software package relative to the behavior of the dapped end beam when subjected to variable loading under the bond deterioration as considered in the experimental examination.

The geometry, boundary conditions, loading, and strength properties of the material test setup were carefully mocked up to resemble the experimental investigation. The analysis model was achieved by the concrete model of 3-dimension (COM 3D). COM3D considers the full model of the reinforcement inside the concrete material to form the RC mesh under the multi-directional fixed crack. The reinforcement can be modeled in discrete or smeared, the authors adopted discrete modeling of the reinforcing bar. In this manner, the reinforcing bars are arranged inside the elements of the RCDEB model. The discrete model resembles the exact arrangement in the experimental approach and offers more stability during the analysis process; the model can resist shear and flexure from applied load hence, behaving like a beam element. Furthermore, the mesh density is considered here to avoid the coarseness of the FEM mesh, which could result in unreliable output at the critical portion of the dapped end (Quadri and Fujiyama 2019). In the nonlinear simulation of the RCDEB, the typical model comprises 4288 elements and 7234 nodes with elastic support designed as simply supported. The modeled case of beam S4 is presented in Fig. 7, with sections A-A and B-B showing the discretization of the bond deterioration in the beam. The elements in red color are the target element for the bond loss. Here, the properties of the PVC hollow pipe, with concrete, elastic, and steel properties presented in Table 2, used for the experimental studies were input into these elements with a bond interface applied between the elements and the embedded reinforcing bars. For the case of beams S1 and S5, the bond loss labeled one in section A-A only was considered. Bond loss labeled 1 and 2 was considered for beam S2, while the bond loss labeled 1, 2, and 3 was considered for beam S3. S0 was considered without the bond loss. Other strengths properties considered for the simulation are the tensile and compressive strengths of concrete for each beam, and yield strength of reinforcement conducted experimentally.

When examining the behavior of fatigue loading on

| Material property | Concrete | Steel | Plastic Material | Interface element properties |
|-------------------|----------|-------|------------------|-----------------------------|
| Initial stiffness (N/mm²) | 2.2×10⁴ | 2.1×10⁵ | 3.3×10⁴ | Shear stiffness (N/mm²) 3×10⁴ |
| Poisson ratio | 0.2 | 0.3 | 0.2 | Normal stiffness (N/mm²) 3×10⁴ |
| Unit weight (kN/m³) | 24 | 78.6 | 14.5 | Friction coefficient 0.8 |
| Tensile strength (N/mm²) | 2.48 | - | 51.7 | |

Fig. 6 Constitutive relation of cracked concrete under high fatigue loading (Maekawa et al. 2006).
reinforced concrete (RC) structure, it is not only essential to consider the effect of concrete and that of steel, but also the bond interaction between the two. Figure 7b indicates the interface element model considered in place of the steel and concrete properties under the Mohr-Coulomb’s frictional law (Fujiyama and Maekawa 2011). Free normal stress is assumed after the separation of the element. Rabbat and Russell (1985) varied the average condition of the frictional coefficient of steel in concrete or grout between 0.57 and 0.7, in this study, frictional coefficient (μ) of 0.8 was adopted on the contact surface of the element with an open-slip mode of the interface assumed to be rigid until the tensile stress normal to the interface surpasses the initial bonding. Hence, separation causes loss of contact and the Mohr-Coulomb’s circle friction is triggered. After re-contact, the transfer of stress restart. This path-dependency is the frictional contact planes in the analysis. Here, the interaction of softened crack shear and gap opening is considered. In this study, a sudden jump from continuum to perfect separation is assumed for simplicity. In the finite element analysis, the plastic material properties surrounding the embedded reinforcing bar are modeled as a unit element with the bond
interface of zero thickness applied between this unit element and the surrounding concrete elements. It is assumed that the behavior of the reentrant portion and the tension zone under the influence of the modeled bond loss when subjected to applied load will bring forth the stress action. The properties of the interface element used are also presented in Table 2.

3.2 Applied loading
Firstly, the beams are simulated under similar conditions as in experimental study. S0 and S5 are subjected to monotonic loading at \( \alpha/d=1 \), where \( d \) is the full depth of the undapped section, and at the midpoint respectively on a set of nodes (left side) in the transverse direction (see Fig. 7). S2 and S3 are subjected to the fatigue fixed pulsating load at \( \alpha/d=1 \), while S1 and S4 are tested on fatigue fixed pulsating load at the midpoint only on nodes in the transverse direction. The cyclic loading entails the load application of 5 million cycles with each loading and unloading of the corresponding nodes at five time steps. Each cyclic loading step was equivalent to 0.003 seconds. Secondly, the authors extend the applicability of the numerical analysis, by adopting the COM 3 system, to the moving load. Experimental investigation is not considered in this study due to the availability of materials and handling. Three transverse nodes were selected for the investigation. In real practical engineering, moving loads like the moving vehicles in traffic contribute a serious degradation to the fatigue life of bridge structures when compared to the fixed pulsating (Matsui 1978). Approximately 2 to 3 order differences in service life reduction have been reported experimentally in RC bridge deck (Deng and Matsumoto 2018; Tang et al. 2014). In addition, a similar reduction was reported when the water coupling effect was considered under cyclic loading conditions (Fujiyama et al. 2011). In conducting the numerical study under the moving load, the direct integral path constitutive model was considered to capture the behavior of the RCDEB under the influence of bond deterioration. The direct path is integrated by applying the acceleration factor, \( \xi \), in increasing magnitude to the nonlinear FE coded in the COM3 (Schenk and Gärtner 2004). Three nodes in the transverse direction were selected, loaded, and unloaded consecutively. The nodes include the one corresponding to the shear span to depth ratio of 1 on the right and left sides and the node at the midpoint of the beam. The same time step applied for the fixed pulsating loading in 0.003 seconds was considered here. This is equivalent to 6.67 m/sec in the real practical consideration under the moving load. In consequence, about 3853 steps of 20 iterations are required for the completion of 5 million cycles in 45 hours. The structural degradation at the macroscopic level when the beam is examined under the applied moving load is demonstrated by tensile cracking, compressive crushing, and other forms of failure of concrete and rupture of reinforcement.

4. Results
4.1 Static loading case
Figure 8 is presented to show the relationship between the load and displacement comparison of the experimental and numerical analysis under the applied monotonic loading. The experimental displacement is the net displacement from the beam’s supports and the calculated value at the loading point, while that of the numerical analysis is obtained from the loading point nodes. The crack pattern of the beam under static loading when loaded at \( \alpha/d=1 \) and at the midpoint is shown in Fig. 9, while the numerical simulation of the strain distribution of the beam in the x-x direction is presented in Fig. 10. The authors generally refer to this direction to determine crack strain propagation of the RC under the analysis approach. Figure 11 presents the experimental and numerical analysis of the load versus the strains generated from the hanger reinforcement and the longitudinal nib reinforcement close to the dominant cracks of the beam. The strain was obtained automatically with strain gauges connected to the data logger in case of the experiment, while the space average of the strain localized RC element of the hanger rebar and the nib rebar was taken in

![Fig. 8 Comparison of load-displacement relationships between experiment and analysis (static case).](image-url)
case of the numerical analysis.

The presence of discontinuity at the ends of the DEB that sits on the supports aggravates the high-stress concentration at the re-entrant portions, which leads to the emergence of cracks. It is apparent from Fig. 8 that the beams exhibit an initial linear-elastic relationship, with departure from linearity taking place from approximately 47 kN when loaded at $a/d=1$ for both experiment and analysis. However, there is an observable early deviation from linearity when loaded at the midpoint on the account of the bond loss, and a little difference in loss of stiffness between the experimental and analytical results.

In the experiment, the first diagonal tension crack was observed at the reentrant corner, for beams S0 (loaded at $a/d=1$ and S5 (loaded at the midpoint), under the static capacity condition at about 60 kN and 64 kN respectively. Under increasing loads, the diagonal crack advanced and increased in width towards the lower part of the loading point bringing about stress redistribution and generating more stresses from the steel. Due to aggregate interlock, the diagonal crack was halted, and another crack that led to the shear cracks developed as the monotonic load was further increased. Figure 9 shows the failure pattern of beams S0 and S5, both of which failed under diagonal tension and shear cracks at the critical sections. The first yield load $P_y$ (equivalent to load at yielding of hanger bar), the ultimate load, deflection at yielding and ultimate failure, and the ductility coefficient (the ratio between the
deflection at failure and deflection at yielding) of the test RCDEBs were examined and presented in Table 3 for all the experimentally test beams. In the case of numerical analysis by adopting the constitutive model, tension cracks are initiated from the reentrant portions in both beams and propagated towards the underside of the loading point. The shear crack developed in beam S5 culminated in the failure of the beam at a capacity of 104 kN. The cracks generated at the bond loss section flow parallel to the plane section of the beam. Figure 10 shows the strain distribution of the beams in the x-x direction. Close conformity is attested to between the experimental and analytical response of the beams S5, however in S0, the failure capacity, as well as the tension stiffening of the experiment, is higher than the analysis, because of the disparity in the crack propagation stage under static loading conditions.

There is a good correlation between the damage response and the failure pattern of the experimental investigation and the numerical approach. An average maximum load of 130 kN and 125 kN was observed in the experiment and the numerical model at the same displacement for beam S0. An average maximum load of 108 kN at 6 mm displacement was observed in the experiment for S5 while the numerical values gave a maximum load of 105 kN at 5.5 mm displacement. The maximum load difference, in this case, is minimal. The validated numerical model can embellish the experimental study and can efficaciously assist in enriching the insight into the problem domain.

The strain generated from the hanger reinforcement, and the longitudinal nib reinforcement close to the dominant cracks for S0 and S5 under monotonic loading were appraised and presented in Fig. 11, for both experiment and numerical analysis. The first diagonal tension crack which occurred at a load up to 60 kN has no significant effect on the reinforcements’ strain. However, when the applied load was increased to about 70% of the first crack load, the strain drastically increased resulting in shear cracks at the reentrant corner of the beam. The stress generated from the propagated diagonal tension cracks from the reentrant corner was mainly absorbed by the intersected hanger reinforcement. The hanger reinforcement of both beams attained their yield strains of 2000 μ at approximately the same capacity. Beyond this, a constant load equivalent to the failure load of each beam produced a dramatic increase in the strain of the hanger rebar. The same strain response is observed when the space averaging of the RC strain element close to the dominant crack is assessed since the whole nonlinearity of the beam relies on the active cracks. The longitudinal strain gauges could have lost contact with the reinforcing bar at high stress, as the reinforcement shows no attainment of yield strain in the experiment. However, there is an appreciable increasing strain response in the analysis over the yield strain.

### 4.2 Cyclic loading case

#### 4.2.1 At midpoint (S1 and S4)

After the static capacity of the RCDEB has been established from the experimental study under the monotonic loading, the stepwise cyclic loading and unloading condition was confirmed. A minimum static capacity load (SC) of 15% was applied and increased by 15% at every
20 cycles until failure. Contact between the beam and the load actuator was guaranteed by loading the beam down to 5% SC. The three-point bending setup and the instrumentation adopted for the static testing of the beam was also employed for the cyclic test.

**Figure 12** shows the relationship between the cyclic load and the vertical displacement for both experiment and numerical investigation. This is a relative displacement obtained by subtracting the vertical displacement at the supports from the measured value at the loading point for the experiment. The strain development around the dominant cracks on the account of the variable cyclic loading has also been assessed and presented. **Figure 12b** shows the cyclic load-strain relationship from the hanger reinforcement while that of the longitudinal reinforcement from the nib is presented in **Fig. 12c** for both experiment and the analysis. The failure pattern for beams S1 and S4 respectively, is shown in **Fig. 13**. Some immature cracks close up when the beams were relieved of the failure load. **Figure 14** shows the strain distribution of the beam subjected to cyclic loading at the mid-point in x-x direction.

On account of the cyclic load in the experiment, the stiffness of the beams degrades gradually with increasing irrecoverable deformation and a gradual rise in strains of the reinforcements as shown by the cyclic load versus displacement response in **Fig. 12a**, cyclic load versus strain of the hanger reinforcement close to the dominant

![Fig. 12 Experiment and analysis comparisons: (a) cyclic load versus displacement, (b) cyclic load versus strain of the hanger reinforcement, (c) cyclic load versus strain of the longitudinal reinforcement.](image-url)
crack in Fig. 12b and the cyclic load versus longitudinal nib reinforcement in Fig. 12c. The damage accumulates with increasing amplitude of the cyclic loading when cycled and partially unloaded to 5% SC. Changes in displacement and strains of the reinforcements are observed to be slow when the applied load is between the range of 15% and 45% SC as the load cycle progresses. Under the influence of the bond loss, there was no observable crack throughout the applied cyclic load of 15% and 30% SC. During the first cycle of 45% SC, hairy cracks were observed on the left sides of the beams but vanished when the beams were unloaded. The stiffness of the beams reduced gradually due to the high stress transfer between the aggregates and the reinforcement. Premature shear crack was observed at 60% SC in the full depth of beam S4 while the previously formed cracks had travelled and extended in width towards the underside of the loading point. Under the same load level, the width of the cracks in beam S1 increased but there was no observable crack propagation. When the cyclic load was increased to 75% SC at the 10th cycle, the diagonal tension crack in beam S1 propagated more and another crack was initiated at the reentrant portion on the right side. Flexural cracks were observed around the area of the bond loss in S4 when cycled to 90% SC (corresponding to an applied load of 86 kN), the shear crack propagated more, and this crack extended upward which consequently resulted in the beam failure. Shear cracks appeared at the 10th cycle in S1 and traveled towards the underside of the loading point. S1 failed at an applied load of 100 kN when intentionally cycled beyond 90% SC. The premature cracks disappeared when the beam was finally unloaded as shown in Fig. 13a. The lower stiffness and capacity displayed by S4 may be due to

![Failure pattern of the test beam under cyclic loading at the midpoint](image)

Fig. 13 Failure pattern of the test beam under cyclic loading at the midpoint 00.

![Strain distribution under cyclic loading condition (x-x direction)](image)

Fig. 14 Strain distribution under cyclic loading condition (x-x direction).
higher bond loss carried when compared with S1 loaded under the same loading conditions. The total of 110 and 127 cycles were recorded for S1 and S4 respectively.

In the analysis, the load at failure, as well as the stiffness was relatively lower with increasing displacement than the experiment when considered at the same condition because of early crack propagation stage and increase in slip action of the rebar. Regardless of this, the hysteresis loop of the cyclic load-strain follows the same trend as the experimental investigation. The reloading and unloading under the direct-path integral system indicate some level of exaggerations when unloaded to 5% SC due to localized damage at the critical section of the beam. Almost the same crack pattern is demonstrated between the experiment and the analysis under the same loading conditions.

4.2.2 At $a/d=1$ (S2 and S3)

Figure 15 indicates the comparisons of the relationship of the cyclic load-displacement, and the cyclic load-strains between experimental investigation and the analysis loaded at shear span to depth ratio of 1. The assessment here follows the same pattern as previously explained. The failure mode of the beams S2 and S3 is presented in Fig. 16. The crack pattern under the analysis demonstrated by the strain distribution along x-x direction is shown in Fig. 17.

Energy is absorbed in the loop as the variable amplitude of the cyclic load is gradually increased until failure. In the experimental investigation, crack was initiated at

![Fig. 15 Experiment and analysis comparison: (a) cyclic load versus displacement, (b) cyclic load versus strain of the hanger reinforcement, (c) cyclic load versus strain of the longitudinal reinforcement.](image)
the recess corner close to the loading point at the 10th cycle for beam S2 and at the second cycle for beam S3 at an applied cyclic load of 30% SC. The crack formed widened when the load was increased to 45% SC in case of S2 which induced diagonal crack on the other side of the reentrant portion at the 8th cycle, causing an increase in dowel shear at this corner. The hanger reinforcement at this section resisted the cyclic load and induced more strain in the steel strain when reloaded as indicated in Fig. 15b. However, lesser strain is generated from the nib longitudinal reinforcement compared to the hanger bar (Fig. 15c). The beam displacement increased gradually at 60% static capacity, the reentrant crack propagated diagonally to the loading point, which increased the slipping action of the RC. When the load was further increased to 75%, the crack traveled vertically downward from the compression zone to the flexural zone. A large crack opening was formed between the recess and the full depth at the reentrant portion; this consequently affected the total stiffness of the beam under the influence of the bond loss. A similar failure pattern was observed in S3 at the same applied load capacity that resulted in the failure of the beam with an observable flexural crack at the nib section. The total of 87 and 95 cycles were recorded for S2 and S3 respectively.

It is ascertained that the crack patterns discovered under the experiment are clearly simulated under the analytical approach although at a relatively lower capacity compared with the experiment. The crack formed traveled diagonally towards the underside of the loading point and then propagated vertically downward towards the critical section of the full depth of the beam before developing a diagonal crack at the flexural zone. The hanger bar close to the dominant cracks bore the stress generating high strain as presented in Fig. 15b. Direct shear crack was observed in the case of S3 together with
concrete crushing at the compression zone. This authen-
ticates the reliability of the analytical mechanism for the
examination of the RCDEBs under the cyclic loading
conditions.

4.2.3 Crack angle inclination
The crack angle inclination to the horizontal of the
dominant crack in each beam was examined. Figure 18
indicates the measurement of the crack angle with a
protractor. The existence of the bond loss between the
cement and the reinforcement rib under the unloading
and reloading states facilitates the crack occurrence to
the propagation state of concrete. Beams S1, S2, S3, S4,
and S5 with the bond loss, carry relatively small crack
angles of 31°, 20°, 16°, 22° and 35°, respectively, com-
pared to the control beam, S0, with a crack angle of 40°.
When the beam is subjected to cyclic loading, there was
an opening between the nib element close to the reentrant
corner and the remaining full depth of S2, S3 and S4,
which has bond loss at the beam as shown in Fig. 18. The
crack orientation is changed, the cracks generated near
the reentrant portion of the RCDEB where the bond loss
is located is resisted by the hanger reinforcement and the
longitudinal nib reinforcement. The inclination of the
crack angle to the horizontal is thus reduced while the
width widened.

4.2.4 Discussion
The crack is generally initiated at the reentrant corner of
the beam irrespective of the loading position due to the
discontinuous sections where high stresses concentrate
and when the static load and the low cycle fatigue were
considered at relatively low capacity i.e., about 15% SC,
the development of crack at any section of the beam was
not prominent for the time being. Additionally, there
hardly any effect on the strains of the hanger and the
longitudinal nib reinforcement in this case. In addition,
when the applied percentage static capacity load was
higher, the hanger reinforcement developed an appreci-
able amount of strain values. The beams tended to have
more cracks that propagated to greater extents with
yielding of stirrups close to the reentrant portion. The
loading was halted before stirrup fracturing for further
investigation.

The experimental results established that the ductility,
the capacity to fail of the beam, the strain develop-
ment, the failure mode, and the time taken to complete
the cyclic loading differ significantly depending on the
loading condition. When the loading positions are com-
pared, the load magnitude that resulted in crack initiation
and propagation under the cyclic loading is different
although of the same initiation point and response at the
initial phase at the same amount of shear reinforcement.
Thus, the shear capacity is affected by the shear span to
depth ratio rather than the overall depth of the beam.
Appreciable failure is recorded at \( \frac{a}{d} = 1 \). Furthermore,
RCDEBs have the tendency to develop an arch action
that offers more resistance during the cyclic loading

![Fig. 18 Inclination of the dominant cracks of experimented RCDEBs and proposed damage pattern of the RCDEB under the bond loss.](image-url)
beyond the diagonal tension crack, which increases the shear capacity and ductility of the beam when loaded at the midpoint (see Figs. 8 and 12a).

There is a tolerable level of conformity between the numerical investigation, under the direct path integral approach based on the time-dependency as discussed hitherto, and the experimental examination under static and cyclic loading. The proportional peak in the strains of the assessed reinforcements due to redistribution of stress at high applied load can be identified with the diagonal tension crack emanating from the reentrant corner and propagating towards the loading point of the beams, as this forms the integral damage of the beam. Although there is a disparity between the load-displacement of experiment and analysis of S4 (Fig. 12a) due to irrecoverable large plastic deformation of the nib corner, the failure patterns accede well with each other. The possible explanation could be linked to the high level of bond loss incorporated which developed high moment under the cyclic loading.

4.2.5 Moving load results
The damage response of the beams extended to the moving type load under the numerical examination is presented in Fig. 19 by the strain distribution, the displacement of the beam under the percentage static capacity loading has also been assessed as indicated in Fig. 20, the oscillation in displacement to number of cycle is as a result of the magnification of the acceleration factor, $\xi$, in the direct path integral system (Maekawa et al. 2006a).

The high cycle fatigue stepwise loading and unloading process was applied to the three selected nodes in the transverse direction. Fatigue applied load of 15%, 30%, 45%, and 60% static capacity was considered. Firstly, the applied fatigue load was tested at the midpoint (labeled 1) under the stepwise loading; it was then unloaded and reloaded on the right (labeled 2) and then point labeled 3. At an applied static capacity of 15%, the cracks were initiated at the reentrant portion of the beams, but at a different number of cycles. The crack propagation is dependent on the level of the bond loss incorporated in each beam. The response under fatigue shows that cracking can occur when the induced tensile stress in concrete reached its ultimate tensile strength around the bond loss. The degree of damage in each beam increased as the cyclic load capacity was increased, depending on the amount of bond deterioration. Moving load typically exhibits reentrant corner damage, with S4 exhibiting the most damage in short time followed by S3, S2, and S1. At minimum percentage static capacity, it takes beam S1 around 100 million cycles to fail, while the number of cycles decreases as the bond loss increases as presented in the displacement versus the number of cycles in logarithmic counts, Fig. 20. The same scenario was observed in all percentage load cases adopted for the moving load, except in 60% SC where the beams fail around 2 to 3 cycles for all beams. S4 shows more damage at a minimum applied percentage static capacity (15% static capacity) load compared with the other beams, its fatigue life cycle reduces to over hundred thousand (100 000). Fatigue life of S2 is below hundred million cycles, while that of S3 reduces below one million cycles (see Fig. 20). Diagonal tension crack at the recess section, shear crack at the full depth of the beams, flexural crack, and compression cracks due to sustained loading were observed in the beams.

Judging from the response of the beam and the level of damage between the low cycle fatigue (fixed pulsating) and the moving load under the bond deterioration, it is obvious that the moving load offered a devastating effect on the RCDEB. The damage was abrupt and concentrated more at the critical portion of the beam at a shorter time, and a relatively lower percentage of static capacity loading than the fixed pulsating loading. The dominant problem is the diagonal tension failure developed at the recess portion. When a moving load case was compared to a fixed pulsating load, Gebreyouhannes et al. (2008) found a substantial reduction in fatigue life of around 2 to 3 orders of magnitude. A similar reduction was observed when reversed cyclic load was considered on RC slab due to slip accumulation around deterioration zones (Gebreyouhannes et al. 2008). In this study, it was discovered that the reduction in fatigue life under moving load is also affected by the degree of bond deterioration of RCDEBs. At minimum applied static capacity of 15%, reductions of the order of 1 to 3 in fatigue life can be seen when increase in bond deterioration of the beam is compared. It is therefore important to devise a strategy for preventing corrosion and damage to RCDEBs subjected to fatigue loading, such as the moving load problem. In addition, more attention should be paid to the reentrant corners by providing more stiffness to increase the overall performance of the beam.

5. Conclusions
1) The response of the reinforced concrete dapped end beams (RCDEBs) has been investigated experimentally and numerically. A total of six beams were tested, two were adopted for static loading, which serves as the basis for determining the variable low cycle fatigue for the other four beams under the bond deterioration between the reinforcing bar and the concrete, in terms of percentage. This was extended to the moving load under the stepwise loading. The first crack always initiated at the reentrant corner with different angles of inclination due to shear slip and dilation under an increase bond loss.

2) The presence of the bond loss toward the recess corner offers more damage to the beam under the applied load. The presence of shear and diagonal tension cracks was evident, indicating the severity of this section under corrosion. There is a considerable reduction in ductility at yield load between S1 and S4 subjected to cyclic loading compared with S5 sub-
ject to static loading at the midpoint, 41% and 19% reduction in the ductility was obtained in S1 and S4 when compared with the ductility of S5. When the ductility of S2 and S3 was checked and compared with S0, 9%, and 48% reduction was obtained.

3) The nib longitudinal reinforcement and the flexural reinforcement adopted were effective in resisting the tensile stresses and keeping the flexural cracks from further opening, but the hanger reinforcement (stirrups) close to the reentrant corners of the beams was not that effective in keeping the crack at the reentrant corner from opening as the stresses in the hanger reached beyond the yield strength, as estimated by the strain gauges mounted on the bars in this region. Hence, more attention should be directed towards this reentrant corner.
4) The damage level of the RCDEB verified numerically under the moving load indicated a serious concern as the position of the moving load dictates when it fails. The fatigue life of the beams is shortened under a relatively lower percentage static capacity compared to the cyclic loading scenario because of the high stress generated in a short time. Damage could be observed at 15% static capacity in moving load, whereas no damage was observed at the same capacity for cyclic load.

References
ACI, (1995). “Building code requirement for structural concrete (ACI Code 318-95).” Farmington Hills, Michigan, USA: American Concrete Institute.
ACI, (2019). “Building code requirements for structural concrete: Commentary on building code requirements for structural concrete (ACI 318R-19).” Farmington Hills, Michigan, USA: American Concrete Institute.
Arya, C., (2009). “Design of structural elements: Concrete, steelwork, masonry and timber designs to British standards and Eurocodes.” 3rd ed. London and New York: Spon Press.
EN-2, (2005). “Eurocode 2: Design of concrete structures (European Standard EN 1992-2).” Brussels: European Committee for Standardization.
CEB-FIP, (1993). “CEB-FIP model code 1990: Design code.” London: Thomas Telford Ltd.
Cook, W. D. and Mitchell D., (1988). “Studies of disturbed regions near discontinuities in reinforced concrete members.” ACI Structural Journal, 85(2), 206-216.
Deng, P. and Matsumoto, T., (2018). “Determination of dominant degradation mechanisms of RC bridge deck slabs under cyclic moving loads.” International Journal of Fatigue, 112, 328-340.
El-Kashif, K. F. and Maekawa, K., (2004). “Time-dependent nonlinearity of compression softening in concrete.” Journal of Advanced Concrete Technology, 2(2), 233-247.
Farreras-Alcover, I., Chryssanthopoulos, M. K. and Andersen, J. E., (2017). “Data-based models for fatigue reliability of orthotropic steel bridge decks based on temperature, traffic and strain monitoring.” International Journal of Fatigue, 95, 104-119.
Fathalla, E., Tanaka, Y. and Maekawa, K., (2018). “Remaining fatigue life assessment of in-service road bridge decks based upon artificial neural networks.” Engineering Structures, 171, 602-616.
fib, (1996). “Practical design of structural concrete.” Lausanne, Switzerland: Fédération Internationale du Béton.
Fujiyama, C., Kobayashi, K., Zhan, J. and Maekawa, K., (2011). “Fatigue life simulation of RC bridge slab with initial defects under water.” Procedia Engineering, 14, 1897-1905.
Fujiyama, C., Tang, X. J., Maekawa, K. and An, X. H., (2013). “Pseudo-cracking approach to fatigue life assessment of RC bridge decks in service.” Journal of Advanced Concrete Technology, 11(1), 7-21.
Fujiyama, C. and Maekawa, K., (2011). “Computational simulation for the damage mechanism of steel-concrete composite slabs under high cycle fatigue loads.” Journal of Advanced Concrete Technology, 9(2), 193-204.
Gebreyouhannes, E., Chijiwa, N., Fujiyama, C. and Maekawa, K., (2008). “Shear fatigue simulation of RC beams subjected to fixed pulsating and moving loads.” Journal of Advanced Concrete Technology, 6(1), 215-226.

Gebreyouhannes, E., Kishi, T. and Maekawa, K., (2008). “Shear fatigue response of cracked concrete interface.” Journal of Advanced Concrete Technology, 6(2), 365-376.

Goto, Y. M., (1971). “Cracks formed in concrete around deformed tension bars.” ACI Journal, 68(4), 244-251.

Hiratsuka, Y., Senda, M., Fujiyama, C. and Maekawa, K., (2016). “Fatigue-based structural behavior of RC bridge slabs with different loading histories.” Journal of Japan Society of Civil Engineers, Ser. E2 (Materials and Concrete Structures), 72(4), 323-342. (in Japanese)

Johnson, P., (2007). “The influence of water on fatigue.” Proceedings of Japan Concrete Institute, 9(2), 627-632. (in Japanese)

Mattock, A. H. and Chan, T. C., (1979). “Design and behavior of dapped-end beams.” PCI Journal, 24(6), 28-45.

Mohammed, B. S., Aswin, M., Liew, M. S. and Zawawi, N. A. W. A., (2019). “Structural performance of RC and R-ECC dapped-end beams based on the role of hanger or diagonal reinforcements combined by ECC.” International Journal of Concrete Structures and Materials, 13(1), Article No. 44.

Moreno-Martínez, J. Y. and Meli, R., (2014). “Experimental study on the structural behavior of concrete dapped-end beams.” Engineering Structures, 75, 152-163.

Quadri, A. I. and Fujiyama, C., (2019). “Fatigue investigation of RC Gerber girder abutment: A case study of the collapse of the De la Concorde (Quebec) Bridge in Canada.” In: Proc. 2nd IABSE Young Engineers Colloquium, Tokyo, Japan 7-8 November 2019.

Rabbit, B. G. and Russell, H. G., (1985). “Friction coefficient of steel on concrete or grout.” Journal of Structural Engineering, 111(3), 505-515.

Reynold, G. C., (1969). “The strength of half-joints in reinforced concrete beams.” London: Cement and Concrete Association.

Schenk, O. and Gärtner, K., (2004). “Solving unsymmetric sparse systems of linear equations with PARDISO.” Future Generation Computer Systems, 20(3), 475-487.

Shakir, M. Q. and Abd, B. B., (2020). “Retrofitting of self compacting RC half joints with internal deficiencies by CFRP fabrics.” Jurnal Teknologi (Sciences & Engineering), 82(6), 49-62.

Suryanto, B., Staniforth, G., Kim, J., Gebreyouhannes, E., Chijiwa, N., Fujiyama, C. and Woodward, P. K., (2019). “Investigating the mechanism of shear fatigue in reinforced concrete beams subjected to pulsating and moving loads using digital image correlation.” In: MATEC Web of Conferences 258: Proc. International Conference on Sustainable Civil Engineering Structures and Construction Materials, Yogyakarta, Indonesia 5-7 September 2018. Les Ulis, France: EDP Sciences.

Syed, Z. I., Kewalramani, M. and Hejah, E. S., (2019). “Structural reliability of dapped end beams with different reinforcement layouts under dynamic loading.” IOP Conference Series: Materials Science and Engineering, 575, Article No. 012003.

Tang, X. J., Fujiyama, C., Shang, F., Maekawa, K. and An, X. H., (2014). “Residual fatigue life assessment of damaged RC bridge slabs based on site-inspection for cracking: A quantitative discussion.” Advances in Structural Engineering, 17(4), 481-494.

Wang, Q., Guo, Z. and Hoogenboom, P. C. J., (2005). “Experimental investigation on the shear capacity of RC dapped end beams and design recommendations.” Structural Engineering and Mechanics, 21(2), 221-235.