FATIGUE BEHAVIOR OF A DEVELOPED UHPC-FILLED PRECAST DECK JOINT IN BULB-TEE BRIDGE GIRDER SYSTEM REINFORCED WITH RIBBED-SURFACE GFRP BARS

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ABSTRACT

This paper presents an experimental investigation of a developed ultra-high-performance concrete (UHPC) filled precast deck joint in bulb-tee bridge girder system reinforced with glass fiber reinforced polymer (GFRP) bars. Two actual-size, GFRP-reinforced, precast concrete deck slabs were erected to perform fatigue tests using the foot print of the Canadian Highway Bridge Design Code (CHBDC) truck wheel loading. Each slab had 200-mm thickness, 2500-mm width and 3500-mm length in the direction of traffic and rest over braced twin-steel girder system. The closure strip between connected precast slabs has a width of 125 mm with vertical shear key. GFRP bars in the precast slab project into the closure strip with a headed end 100 mm. Two types of fatigue tests were performed, namely: (i) accelerated variable amplitude cyclic loading and (ii) constant amplitude cyclic loading, followed by loading the slab monotonically to-collapse. Overall, the test results demonstrated the excellent performance of the developed closure strip details.

Keywords: GFRP-bars, Bridge deck slabs, Prefabricated girders, Closure strips, Fatigue, Ultra-high-performance concrete.

INTRODUCTION

The use of non-corrosive glass fiber reinforced polymer (GFRP) to replace reinforcing steel in deck slabs of precast bulb-tee girders is considered an innovative solution to eliminate the deterioration of deck slabs due to corrosion of steel. GFRP reinforcing bars has many advantages over steel reinforcement, such as a non-corrosive nature, high tensile strength, durable and light weight. A Prefabricated bridge system made of deck bulb-tee (DBT) girders, shown in Fig. 1, can be an attractive choice for accelerating bridge construction. In this system, the concrete deck slab is cast with a prestressed girder under controlled conditions at a fabrication facility and then transported to the bridge site. This system requires that longitudinal deck joints be provided to transfer the load between adjacent units. Prefabricated elements, which can be quickly assembled, could reduce design efforts, negative impact on the environment in the vicinity of the site, lane closure times and inconvenience to the traveling public.

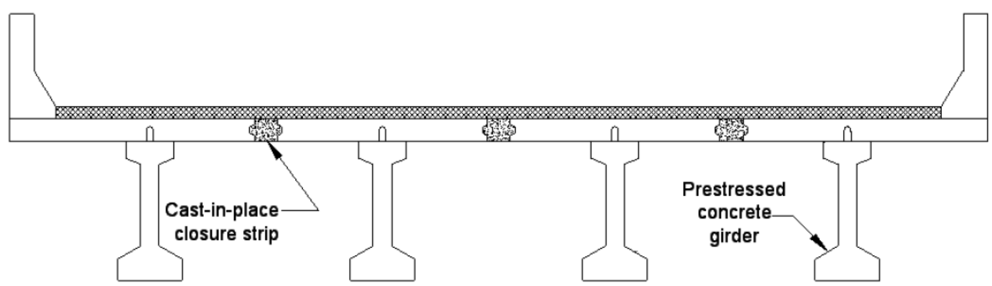


Fig. 1 Prefabricated bridge superstructure made of bulb-tee girders

Development of UHPC-filled precast deck joint in DBT girders system reinforced with GFRP bars provides advanced high performance connection technique for accelerating bridge replacement. This system combines the advantage of the non-corrosive nature of GFRP reinforcing bars with the superior properties of the UHPC, such as high strength, increased bond capacity and durability. Because this is relatively new technology, the Canadian Highway Bridge Design Code, CHBDC[1], and AASHTO-LRFD Bridge Design Specifications[2] do not provide guidance to design prefabricated concrete girder/deck joints made with GFRP bars. Also, there is not enough information available in the literature to design such joints, nor is there test data available to give confidence to the design of such joints. Most recently, a few authors developed and tested to-collapse joint details between flanges of precast bulb-tee girders with projecting straight/U-shaped/headed steel bars (among them: Shah et al.[3,4]; Au et al.[5]; Li et al.[6]; Au et al.[7]; Graybeal[8]). A Literature survey showed that limited number of experiments conducted on bridge deck slabs to examine their fatigue and ultimate load carrying capacity under wheel loads (among them: Sonoda and Horikawa[9]; Pardikaris and Beim[10]; Mufti et al.[11]; Matsui et al.[12];Mufti et al.[13]; Graddy et al.[14],El-Ragaby et al.[15]). This paper presents an investigation of fatigue behavior of a developed UHPC-filled precast deck joint in DBT girders system reinforced with GFRP bars. Two full-scale bridge deck slab models were built to perform fatigue tests to determine their behavior under CHBDC truck wheel loading. The following sections summarize the specimens' details, test setup, experimental test procedure and test results.

DESCRIPTION OF PRECAST BRIDGE DECK SLAB MODEL

GFRP bars with headed-end and straight part used in this study have a tensile strength of 1188 MPa and modulus of elasticity of 64 GPa. The special ribbed-surface profile of these bars, shown in Fig. 2, ensures optimal bond between the concrete and the bar. The head of 16 mm diameter bar used in this study is approximately 100 mm long, with outer diameter of 40 mm (2.5 times the diameter of the bar) tapers in five steps to the outer diameter of the blank bar. This geometry ensures optimal anchorage forces and minimal transverse splitting action in the vicinity of the head. Given the GFRP's small transverse strength and relatively low modulus of elasticity, the shear strength of GFRP reinforced deck slab is lower than that for steel-reinforced deck slab. However, this issue is not important since shear strength in deck slabs is provided by concrete only.

Two full-scale precast bridge deck slab models were designed according to CHBDC specifications were tested in this study. Each slab was formed of two identical precast slab panels 3500x1187.5x200 mm, with a 50-mm deep, 40-mm wide, trapezoidal shape shear key throughout the girders length. The precast slabs represent the flange portions of adjacent bulb-tee precast pretensioned concrete girders shown in Fig. 3 (a). A 125-mm wide closure strip was introduced between the precast flanges of the bulb-tee girders as shown in Fig. 3 (b). The precast slab bottom GFRP bars projected into the joint with headed end to provide a 100-mm embedment length in the tension zone of the slab thickness, while the top transverse GFRP bars projected into the joint with a 100-mm embedment length in the compression zone of the joint. It is assumed that DBT girders will be aligned to provide 125 mm gap that can be filled using UHPC having minimum specified strength of 100-MPa. It should be noted that Fig. 3 (b) shows projecting GFRP bars from one side of the joint only for clarity and the joint would consist of staggered projecting bars that would allow for ease of assembly in the bridge site.

EXPERIMENTAL PROGRAM

The experimental program included two full-scale deck slab specimens of 200 mm thickness, 2500 mm width and 3500 mm length in the direction of traffic. The deck slabs were supported over two W610X241 steel beams of span 7,000 mm and transverse K-bracings at their ends. The clear spacing between the supporting beams is taken as 2000 mm. The precast slabs and the supporting beams were made fully composite with shear connector pockets and shear studs.

The deck slabs were reinforced with GFRP bars per the reinforcement ratio specified in CHBDC. The deck slabs were formed considering the precast deck system shown in Fig. 1 and the precast flange-to-flange connection detail shown in Fig. 3 (b). Headed end GFRP bars of 16-mm diameter, spaced at 140 mm c/c, were used as bottom tension reinforcement while the top main and transverse reinforcements were 12-mm GFRP bars spaced at 200-mm c/c. Transverse bottom reinforcement in the tested slabs were provided by 16-mm diameter GFRP bars, spaced at 225-mm c/c. Bottom and top concrete covers of 38 mm were used for the deck slab. Figure 4 shows sequence of construction and assembly of the precast jointed slabs.

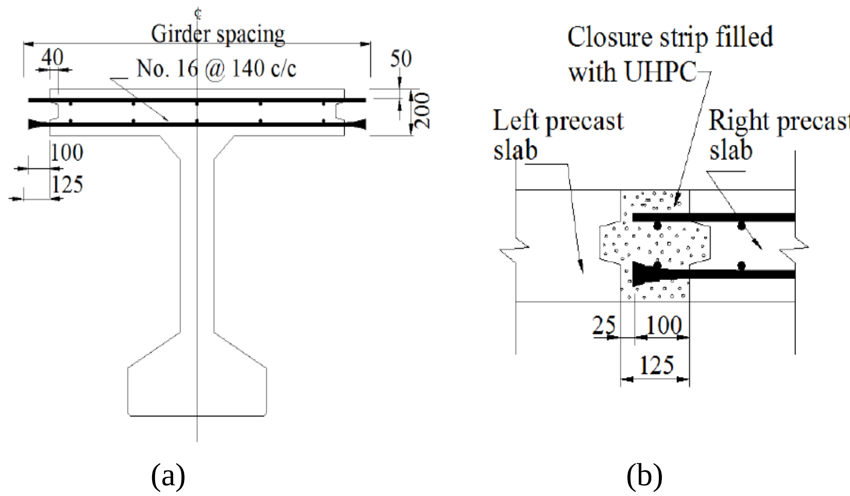
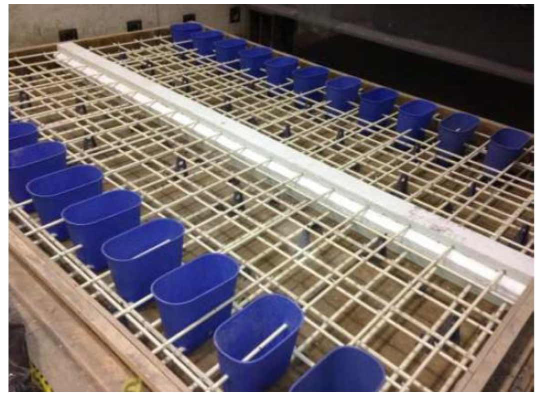


Fig. 2 Views of GFRP bars

Fig. 3 Schematic diagrams of: (a) DBT-girder and GFRP-bars; (b) Proposed closure strip

Concrete having a specified 28-day compressive strength of 35 MPa was used for precast deck slabs. Standard cylinders of 150-mm diameter and 300-mm height were cast concurrently with the casting of the deck slabs. An average of three cylinders were cast and stored close to the test samples to ensure the same curing conditions after casting. Ultra-High-Performance Concrete (UHPC) having a 28-day specified strength of 100-MPa was used for closure strip. Non-shrink grout extended with 9.5-mm pea gravel was used for shear pockets. The grout has a specified 3-day compressive strength of 31 MPa and 28-day strength of 59 MPa. During the pouring of the UHPC/grout into the precast deck slab closure strips/shear pockets, standard cylinders of 100-mm diameter and 200-mm height were cast and kept close to the test samples. The average compressive strengths of the tested concrete deck slabs S1 and S2 were 51 and 53.8 MPa, respectively, while the UHPC average compressive strengths for S1 and S2 were 183.2 and 192 MPa, respectively.

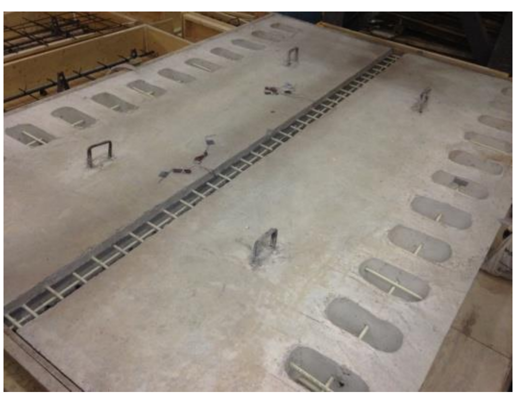
a) Lower layer of GFRP bar at joint



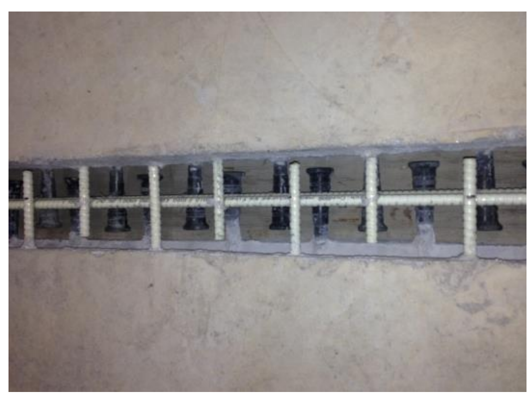
b) GFRP bars and sytrofoam forming the joint

c) Casting the slab

d) Projecting GFRP bars after removal of the styrofoam



e) Precast slab assembly



f) Close-up view of GFRP at the closure strip

g) Close-up view of dropping UHPC into the joint

h) After Filling closure strip with UHPC

Fig. 4 Sequence of construction and assembly of the precast jointed slabs

TEST SETUP AND INSTRUMENTATION

All precast slabs were tested under a 250x600 mm single patch load at the center of their clear span. This patch load is equivalent to the foot print of CHBDC wheel load of 87.5 kN. A 50-mm thick steel plate was used to transfer the load to the bridge deck slab; neoprene pad was used to ensure an even distribution of the load pressure on the contact area of the deck slab surface. To apply partial restraint to the slab ends over the girders, the pair of precast slabs and the supporting girders were made fully composite with shear connector pockets and shear studs as shown in Fig. 5 (a) through (c) . The steel girders were simply-supported over steel pedestals with a clear span of 7,000 mm. Elastomeric pads of 300x300x25 mm, were placed between steel pedestals and supporting girders to ensure boundary conditions were achieved. A 500 kN capacity with 250 mm stroke hydraulic actuator, was used to apply the fatigue loads. Figure 5 (d) shows the experimental setup used for testing the deck slab specimens.

The structural response during the cyclic loading or the static loading of the deck slabs was captured through the use of electronic instrumentation. Potentiometers and Linear variable differential transducers (LVDTs) were used to measure deflections at specified locations of the deck slabs. Electrical-resistance strain gauges were installed on the GFRP bars and the top surface of the deck slab around the loaded area to monitor the strain in the rebar and concrete. Crack displacement transducers were set across the cold joint between the interface of precast panel and the closure-strip to measure the crack width. All instrumentation devices were connected to an electronic data acquisition system (5000) for monitoring and data recording.

a) Twin-girder assembly and shear connectors b) Filling shear pockets with grout

c) Precast jointed-slab before loading d) Test setup for fatigue testing

Fig. 5 Sequence of precast slab and steel girder assembly and test setup

CYCLIC LOAD TESTS

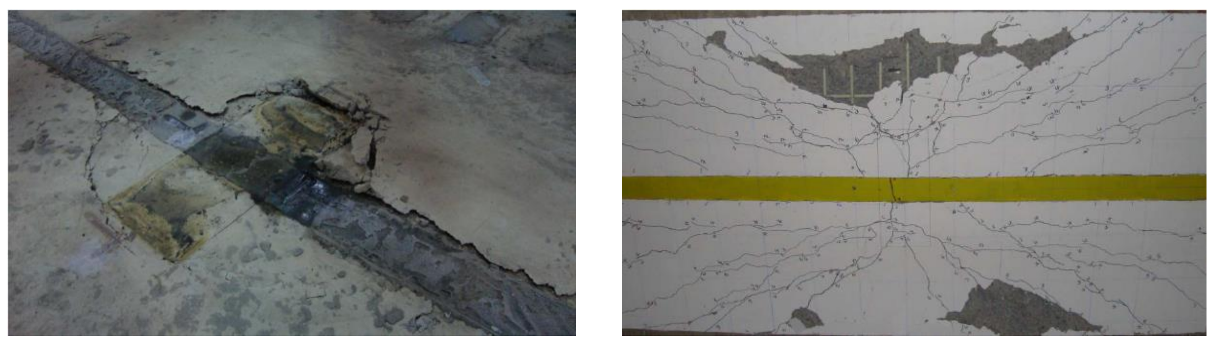
In this research, two fatigue loading schemes were used, namely: accelerated fatigue loading with variable amplitude fatigue (VAF) loading and constant amplitude fatigue (CAF) loading. Prior to starting fatigue load tests, each slab was pre-cracked by performing static load test up to 1.5 times the fatigue limit state (FLS) loading of 183.75 kN and unloaded to zero. This test was conducted first to determine the cracking load and initiate cracks to simulate real bridge state of stress. In the VAF loading test, the slab was subjected to sinusoidal waveform fatigue load cycles between a minimum load level and variable maximum load levels. The minimum load level was set at about 15 kN, representing the effect of superimposed dead loads on bridge deck (i.e. asphalt and insulation layers). This minimum load level will assist in preventing any impact during cyclic loading. Different peak load levels were selected as multiples of the fatigue limit state (FLS) loading as specified in CHBDC. The CHBDC FLS load is specified using the maximum wheel load of 87.5 kN with 40% dynamic load allowance and a FLS live load factor of 1.0. This leads to a FLS load of 87.5x1.4x1.0 = 122.5 kN, according to CHBDC Clause 3.5.1. In this research, a maximum peak load levels of 1.0, 1.5, 2.0, 2.5, 3.0, 3.5 and 4, which correspond to 122.5, 183.75, 245.00, 306.25, 367.50, 428.75 and 490.00 kN, respectively. Each maximum peak load level was applied for 100,000 cycles at frequencies of 2 Hz or less, depending on the stiffness of the specimen. It should be noted that at the end of each maximum peak load level, a static load test similar to the pre-cracked static load test was conducted to assess the degradation that may occur in the deck slab due to fatigue loadings. This VAF loading was applied to slab, S1.

A CAF loading test, followed by loading the slab monotonically to-collapse, was applied for deck slab, S2. In this loading test, constant amplitude of load, representing the FLS load specified in CHDBC of 122.5 kN, was applied at a frequency of 4 Hz for 4 million cycles. Similar to the VAF, but at the end of each 250,000 cycles at a specified load level, a static load test was conducted. After completing the cyclic loadings of 4 million cycles, a static load to failure was applied to slab, S2. The static loading was completed through the use of manually operated hydraulic jack and the load was applied in monotonic increments with temporary holds occurring at approximately 50 kN intervals to allow for inspection of crack initiation and propagation.

TEST RESULTS

This section discusses the structural behavior of the tested specimens in the form of crack pattern, slab vertical deflection and ultimate load carrying capacity. The first deck slab, S1, was tested under VAF loading and the test was completed according to the process described previously. Figure 6 shows the crack pattern at failure on the top and underside of the precast deck slab, S1. Prior to starting the fatigue load test, the slab was pre-cracked by applying a static load of 183.75 kN. It was observed that first hairline cracks were formed at the cold joint between the precast concrete and the closure strip. After applying the first 100,000 cycles at a peak load of 122.5 kN and frequency of 2 Hz, these fine cracks started to widen gradually and new transverse cracks were propagated at mid-span under the loaded area. With increasing load cycles, more cracks were developed in the longitudinal (parallel to the supports) and radial directions. After completing 100,000 cycles at peak load of 245 kN, a transverse crack was observed across the closure strip at mid-span under the loaded area. After that the deck slab started to undergo drastic decrease in flexural stiffness and the remaining peak loads were completed with reduced frequencies. It should be noted that the maximum peak load reached at the final peak load was 475 kN at frequency of 0.5 Hz. The deck slab,S1, failed under punching shear and failure occurred at peak load of 475 kN and after completing 616,145 cycles. Figure 8(a) shows the static load-deflection relationships of slab, S1, after different fatigue loading steps.

The second deck slab, S2, was tested under CAF loading and the test was completed according to the process described previously. Slab, S2, was subjected to 4,000,000 cycles at a peak load of 122.5 kN and a frequency of 4 Hz. After completing the cyclic load, the slab didn't fail and a static test was applied until failure. The slab, S2, failed at maximum load of 758 kN and maximum deflection of 22.85 mm. The failure was due to punching shear as shown in Fig. 7. Figure 8(b) shows the static load-deflection relationships of slab, S2, after different fatigue loading steps. Figure 9 depicts the static load-deflection relationship of, S2, to failure.
It should be noted that the term “S” in Fig. 8 refers to static loading following the completion of each fatigue loading step, while the number following “S-” refers to the peak load at which the slab completed 100,000 cycles in case of VAF or number of completed cycles in case of CAF. For example, S-122.5 in Fig. 8(a) refers to the static loading and unloading cycle after completing 100,000 cycles at 122.5 kN peak load and S-250,000 in Fig. 8(b) refers to the static loading and unloading cycle after completing 250,000 cycles.

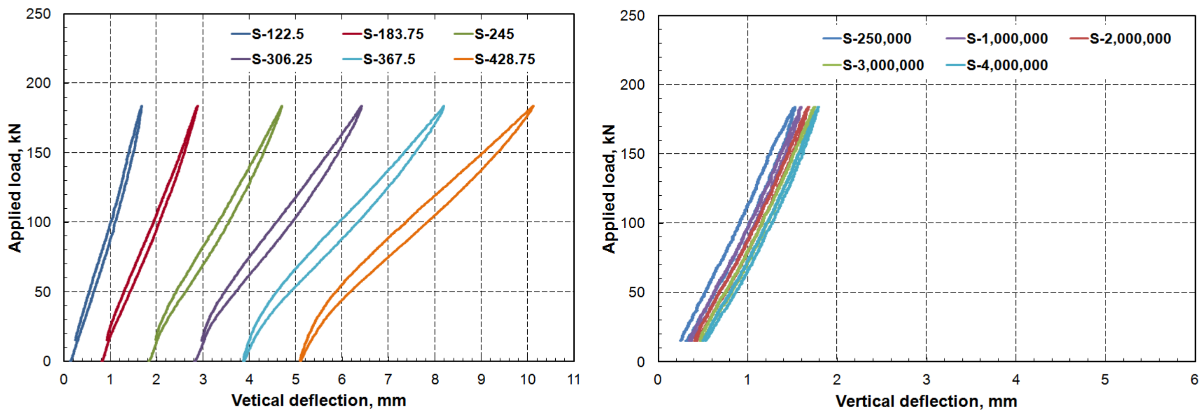


a) Slab top surface at wheel load location b) Slab bottom surface with closure strip

Fig. 6 Views of punching shear crack pattern and failure of precast jointed slab S1

a) Slab top surface at wheel load location b) Slab bottom surface with closure strip

Fig. 7 Views of punching shear crack pattern and failure of precast jointed slab S2



a) Slab S1 b) Slab S2

Fig. 8 Static load-deflection relationships for tested slabs after different fatigue loading steps

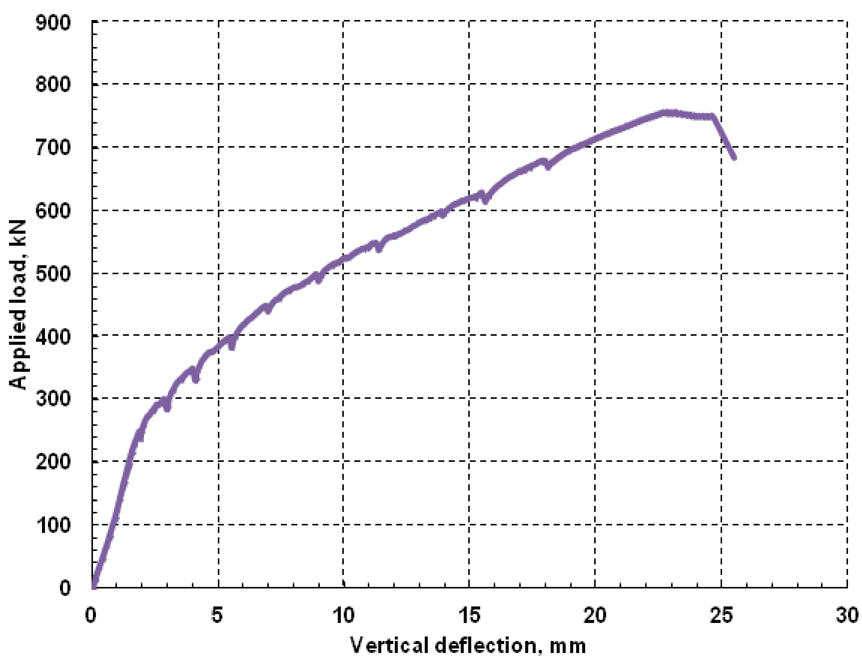


Fig. 9 Static load-deflection relationship obtained at mid-span for slab S2

CONCLUSIONS

This paper investigates the fatigue behavior and fatigue life of a developed UHPC-filled precast deck joint in DBT girders system reinforced with GFRP bars when subjected to CHBDC wheel loading. Based on the experimental results, it can be concluded that the GFRP bars with headed ends can provide a continuous force transfer in the longitudinal joint for deck bulb-tee bridge systems while reducing the closure strip width to accelerate bridge construction. Experimental results also indicate that the GFRP-reinforced deck slab showed high fatigue performance and there was no fatigue damage when subjected to 4,000,000 cycles under FLS load of 122.5 kN specified in CHDBC.

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