Research Article

Fatigue Performance of Steel–UHPC Composite Bridge Deck System with Large Longitudinal Rib

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1.Introduction

Orthotropic steel decks (OSDs) have been widely used in bridges due to its lightweight, easy to assemble, and high bearing capacity [1, 2]. However, the fatigue cracking, which significantly reduces the service quality and severely impairs the durability of OSDs, is prominent and frequent [3, 4]. The fatigue cracking of OSDs is mainly caused by combined action of intrinsic and extrinsic factors. The former includes manufacturing defects, weld residual stress, and weld geometry, the latter is the traffic loads [5, 6]. The fatigue cracks at the rib-to-deck joints, rib-to-diaphragm joints, and the cope holes in the diaphragm are shown in Figure 1.

Scholars have made considerable efforts to promote the fatigue behavior of OSDs [7–10]. On the one hand, the fatigue resistance of OSD can be enhanced by increasing the deck thickness and improving the welding techniques such as double-side weld [7, 8]. On the other hand, the fatigue resistance of OSD can be significantly extended using the ultra high performance concrete (UHPC) to form a composite deck, which increases the overall stiffness of the deck system and reduces the nominal stress range of fatigue-prone details without significant increases in the dead weight [9, 10]. Recently, a steel–UHPC composite bridge deck (Figure 2) composed of an OSD with large longitudinal ribs and UHPC layer was proposed for further optimization [11–13]. The large longitudinal rib has opening widths of 400 mm to 450 mm and heights of 300 mm to 330 mm. Compared to the traditional U-rib, it reduces the length of rib-to-deck weld seam and the associated welding cost by 30% to 40%, then the risk of fatigue cracking of rib-to-deck joint is decreased [11, 12]. In addition, the large longitudinal
rib can also increase the flexural stiffness of the deck system in longitudinal direction, thereby alleviating the tensile stress of the UHPC layer in the negative moment region under the vehicle load [13].

To use a steel–UHPC composite bridge deck as a fatigue-resistant technology for OSDs, the fatigue performance of OSDs as well as the cracking behavior and fatigue resistance of the UHPC layer must be evaluated. Qin et al. [14] and Abdelbaset et al. [15] studied the effect of UHPC layer on the stress state of fatigue-prone details of OSDs through on-site measurement and full-scale model test, respectively. The results showed that in steel–UHPC composite bridge deck, the stress range in rib-to-deck joint below the endurance limit, while that of the rib-to-diaphragm and rib splice are still higher than the endurance limit. Feng et al. [16], Lu et al. [17], and Wei et al. [18] investigated the cracking behavior of the UHPC layer under static and cyclic load, and it is concluded that the cracking of the UHPC layer was inevitable and the crack width will increase under the action of the cyclic load. Liu et al. [12] conducted a fatigue test on a full-scale steel–UHPC composite bridge deck model, observing that the UHPC layer cracked under the design fatigue load of approximately 200,000 cycles. The experience with the use of UHPC composite structures over time indicates that the cracking width and number of cracks is mainly influenced by the fiber content and reinforcement ratio [19, 20]. However, studies on the fatigue resistance of composite bridge decks with large longitudinal ribs are limited. In addition, the use of a large longitudinal rib leads to an increase in span of the deck plate. Typically, this increase generates considerable tensile stress in the transverse direction of the steel–UHPC composite bridge deck, rendering the deck cover prone to fatigue failure under external loads. Therefore, a study of the fatigue performance and damage accumulation process of steel–UHPC composite bridge decks with large longitudinal rib is necessary.

This research aims to evaluate the fatigue performance of steel–UHPC composite bridge deck with large longitudinal ribs. First, two types of specimens were tested to assess the failure mode and degradation mechanism of the deck system along the longitudinal and transverse directions. The load-deflection, load-strain, and cracking behaviors of specimens under cyclic loads were studied by fatigue test; then, the fatigue resistance of the composite bridge deck with a large longitudinal rib was evaluated. In addition, the effects of fatigue damage on the UHPC layer and studs with respect to the fatigue performance of the steel–UHPC composite bridge deck were investigated.

### 2. Experimental Program

#### 2.1. Test Specimens

As illustrated in Figure 2, the mechanical behavior of the OSD (or UHPC layer-stiffened OSD) exhibits directional anisotropy due to different stiffness properties in the orthogonal directions [21]. Generally, three structural component systems must be considered in the design and analysis of OSDs: (1) System I—the deck as part of the main carrying member, (2) System II—the stiffened steel deck consisting of longitudinal ribs, transverse diaphragms, and the deck plate, (3) System III—the deck plate supported by the rib wall of longitudinal ribs. After the introduction of the UHPC layer as part of the deck, the stiffness of the orthotropic bridge deck in stress systems II and III significantly improves. Under vehicle loads, the UHPC above the transverse and longitudinal ribs will be in tension when UHPC participates in the bridge deck system under systems II and III. Therefore, the experiments have...
been conducted on two specimens designated as transverse and longitudinal specimens according to the stress characteristics of the composite bridge deck in systems II and III. The dimensions of the transverse and longitudinal specimens are shown in Figures 3 and 4, respectively. The specimens have the same transverse section with a width and depth of 2160 and 770 mm, respectively; however, their lengths differ, i.e., 900 and 7000 mm for the transverse and longitudinal specimens, respectively. The thickness of the deck plates of the specimens is 14 mm. The diaphragms are 16 mm thick and spaced at 3000 mm. The height, top width, and thickness of the U-rib are 330, 400, and 8 mm, respectively. The 70 mm-thick UHPC layer was poured on the deck and connected to the OSD through headed studs with a 200 mm spacing. The height and diameter of the studs are 50 and 16 mm, respectively. The diameter of steel rebars spaced at 70 mm is 12 mm. The fabrication process of the specimens is shown in Figure 5.

2.2. Material Properties. The steel used for the test specimens were fabricated from Q345qD steel with a nominal yield strength of 345 MPa [22]; HRB400 grade steel rebars were used in the reinforcing mesh. The yield and ultimate strengths of the steel rebars were 400 and 570 MPa, respectively [23]. Steel fibers with diameter, length, and volume ratio of 0.2 mm, 13 mm, and 2%, respectively, were employed in UHPC. After 24 h of natural curing, the castings were maintained at 80°C with 90% humidity for 3 days. Table 1 lists the UHPC ingredients and mix proportion. The mechanical properties of UHPC are summarized in Table 2. The test methods are consistent with techniques employed in previous research [24, 25].

2.3. Test Setup, Instrumentation, and Loading Protocol. The field test setup of specimens is shown in Figure 6. The test specimens were connected with supporting beams fixed on the ground. A servo-hydraulic loading system with a load capacity of 1000 kN was used to apply the cyclic load. The loading position of the transverse specimens is shown in Figure 3. The two loading positions are symmetric with respect to the center of the specimen. The middle spacing is 760 mm, and the single loading area is 270 mm (longitudinal direction) × 320 mm (transverse direction) [26]. The loading position of longitudinal specimens is shown in Figure 4. The two loading positions are symmetric with respect to the middle diaphragm. The middle distance is 1200 mm, and the single loading area is 200 mm (longitudinal direction) × 600 mm (transverse direction) [27].
For the rib-to-deck joints on the transverse specimen, strain gauges are fixed to the bottom side of the deck plate and arranged along the weld root to measure the transverse strain distribution, as illustrated in Figure 7(a). The strain gauges are placed close to the weld toe of the rib-to-diaphragm joints of the longitudinal specimen, as shown in Figure 8(a). Two displacement transducers (DTs) were fixed under loading positions to obtain the deflections of the specimen.

Figure 4: Geometry of longitudinal specimen (unit: mm): (a) cross section view; (b) side view.

Figure 5: Details of specimen fabrication: (a) welding of OSD and studs; (b) installation of rebars and formworks; (c) measuring point arrangement; (d) casting of UHPC; (e) air curing; and (f) steam curing.
specimens (Figures 3(a) and 4(b)). A major problem encountered in the use of a steel–UHPC composite deck was UHPC layer cracking. During the fatigue test, the crack distribution on the surface of the UHPC layer was monitored and recorded through visual inspections. To measure the crack width, 8 and 10 PI gauges for the transverse and longitudinal specimens were attached to the UHPC layer, respectively. The arrangements of PI and strain gauges for the specimens are shown in Figures 8(b) and 9(b), respectively.

The steel rebar mesh is mainly used to test stress and its distribution along the longitudinal direction of the bridge. This stress indirectly reflects the deterioration characteristics of the concrete layer. A pair of strain gauges attached to the opposite sides of the headed studs was used to measure the bending strain, and six strain gauges were utilized to measure the steel rebar strain on the transverse specimen (Figure 7(c)). A total of 40 resistance strain gauges are arranged in 8 rows and 5 columns on the longitudinal rebars of the steel mesh; they are symmetric across the diaphragm section on the longitudinal specimen, as shown in Figure 8(c).

Three cyclic loading stages, labeled as Stages I, II, and III, were executed sequentially with a constant amplitude load range. The minimum load was set at 10 kN and remained constant during the entire experimental process. Table 3 summarizes the applied cyclic loads to each stage.

The fatigue load amplitude of the tests in Stage I is calculated according to the standard wheel weight of 60 kN and impact coefficient of 1.3. Wheel type C was selected for transverse specimens, as suggested in [26], and standard fatigue model III was selected for longitudinal specimens based on literature [27]. The positions of the applied loads $A = \pi r^2$ are shown in Figures 3 and 4. The fatigue load applied to the destructive test in Stage II is based on the first stage load considering an overload factor of 1.5. The upper limit of the first stage test load is increased. If the number of cycles continues to increase up to $4 \times 10^6$, the specimen does not undergo fatigue failure, and the accelerated failure test is implemented. That is, considering an overload factor of 2.0 based on the load in Stage II, the upper limit of the test load in this stage is increased until the number of cycles reaches $6 \times 10^6$. The list in Table 3 indicates that the objective of Stage I is to verify the durability of the composite bridge deck under fatigue. The other stages are implemented to evaluate the fatigue life and failure mode of the specimens.

A static loading test was conducted to acquire the initial mechanical properties of each specimen before they are subjected to fatigue loading. The test was periodically conducted after the completion of a certain number of cycles to record the deterioration process of the test specimens.

<table>
<thead>
<tr>
<th>Table 1: Mix proportion of UHPC (unit: kg/m³).</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
</tr>
<tr>
<td>--------</td>
</tr>
<tr>
<td>634</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table 2: Material properties of UHPC.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive strength (MPa)</td>
</tr>
<tr>
<td>120.3</td>
</tr>
</tbody>
</table>

Figure 6: Experimental setup: (a) transverse specimen; (b) longitudinal specimen.
Figure 7: Instrumentation of transverse specimen (unit: mm): (a) deck strain; (b) UHPC strain; and (c) headed stud strain and rebar strain.
3. Results

3.1. Transverse Specimen

3.1.1. Failure Mode. The distribution of cracks on the UHPC layer is shown in Figure 9. When the number of loading cycles reached 2,000,000, visible cracks were not observed on the UHPC layer in Stage I. The strain in the UHPC layer of the test specimens was stable, and the strain was lower than the elastic limit, which was in the elastic state of the UHPC material. When the loading cycles reached 2,050,000, the initial crack (with a maximum width of 0.01 mm) developed on the UHPC layer surface above the joints of the rib, deck, and diaphragm in Stage II. The crack distribution and width remained virtually constant for the remaining 1,950,000 cycles in Stage II. The maximum crack width at the end of Stage II was approximately 0.02 mm; it did not exceed 0.05 mm. The results show that the crack resistance of the UHPC layer in the test specimen under fatigue load can satisfy the design requirements [19, 20].

Due to the low stress range in the details of the OSD that are not vulnerable to fatigue under the first two loading stages, the fatigue test in Stage III was implemented using a fatigue load that was 2.0 times that in Stage II to accelerate the fatigue damage to specimens. After the cracks appeared, distinct cracks propagated along the longitudinal direction of the OSD near the joints of the rib, deck, and diaphragm. The propagation of cracks was stopped after completing the fatigue tests.
at this stage; many cracks virtually penetrated the specimen. Cracks appeared on the side of the UHPC layer at the edge of the specimen until the total number of loading cycles reached 4,050,000; the maximum crack width was approximately 0.55 mm. At the end of Stage III, the maximum crack width reached 0.06 mm, exceeding the maximum crack width of 0.05 mm under durability control. The variation laws of strain in UHPC during the test are shown in Figure 10.

The variation in rebar stress under fatigue loading is shown in Figure 11. The stress in transversely loaded rebars in the concrete layer remains stable in Stage I. In Stage II, with the appearance of cracks, the stress in the rebars increased to a certain extent; however, it swiftly became constant. With the crack propagation in the UHPC, stress redistribution occurred on the UHPC layer, and the surface stress was continuously transferred to the rebars. The rebar stress increased with the number of fatigue cycles under Stage III.

By monitoring the strain distribution around the welded joint, monitoring the fatigue crack initiation and propagation process also became possible [28]. When the number of fatigue cycles reached 4,250,000, strain gauge U2-DS-LC indicated a sharp drop in strain (Figure 12), indicating that a fatigue crack initiated at the strain measurement point. The variation in strain distribution along the weld root during the fatigue test is shown in Figure 12. The strain redistribution process shows that the fatigue crack propagation is symmetric relative to the central section of the diaphragm. The fatigue crack formation on the welded joint as reported in literature is shown in Figure 13 [12]. The crack initiates at the root of the rib-to-deck and diaphragm joint and extends to the deck thickness.

The variation laws of bending strains in the studs are shown in Figure 14. The strains were stable at the beginning of the fatigue test, indicating that the mechanical properties of the studs had no distinct degradation. The stress redistribution during crack propagation increased the bending strain in the studs as strains rapidly increased in Stage III. The cyclic load was terminated at 6,000,000 cycles. The failure process of the transverse specimen can be

### Table 3: Fatigue loading protocol.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Loading stage</th>
<th>Load level (kN)</th>
<th>Load range (kN)</th>
<th>Loading cycles ($ \times 10^4$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transverse</td>
<td>I</td>
<td>10–166</td>
<td>156</td>
<td>200</td>
</tr>
<tr>
<td></td>
<td>II</td>
<td>20–255</td>
<td>234</td>
<td>200</td>
</tr>
<tr>
<td></td>
<td>III</td>
<td>20–490</td>
<td>470</td>
<td>200</td>
</tr>
<tr>
<td>Longitudinal</td>
<td>I</td>
<td>10–166</td>
<td>156</td>
<td>200</td>
</tr>
<tr>
<td></td>
<td>II</td>
<td>20–255</td>
<td>234</td>
<td>200</td>
</tr>
<tr>
<td></td>
<td>III</td>
<td>20–490</td>
<td>470</td>
<td>200</td>
</tr>
</tbody>
</table>
summarized as follows: (1) multiple cracks form in the UHPC layer; (2) weld root–deck crack starts and propagates in the rib-to-deck joint; and (3) damage initiates and accumulates in the headed studs.

3.1.2. Mechanical Response. During the first two stages, the displacements were fundamentally the same and remained constant; in Stage III, the displacement continued to increase. The displacements versus the number of cycles are described in Figure 15. The increase in displacements indicated that damage occurred in the headed studs and...
UHPC layer, leading to a reduction in transverse stiffness. The variation law of local stiffness at the loading positions is shown in Figure 16.

The fatigue endurance of the OSD must surpass 2 000 000 cycles under specific load models in accordance with current design codes [29, 30]. The transverse specimen underwent 2 000 000 cycles without evident fatigue damage and performance degradation. Due to the excellent crack width control capability of UHPC, the test specimen also satisfied the crack width limitation. Based on the foregoing, the transverse specimen satisfies the design requirements in terms of fatigue and durability. However, the influence of fatigue damage on the mechanical behavior of test specimens is more evident; this damage is caused by UHPC cracking and stud fracture. The addition of loading cycles (Stages II and III) decreases the transverse rigidity of the transverse specimen and increases the peak strain in welded details.

3.2. Longitudinal Specimen

3.2.1. Fatigue Failure Mode. After the longitudinal specimen underwent 750 000 cycles, the initial crack in the UHPC layer occurred at the region above the intermediate diaphragm (Stage I in Figure 17). When the number of loading cycles was increased, multiple cracks were observed between the two loading positions and extended horizontally along the specimen. During the early stage of each load increase, many small cracks were observed. When the load increase reached Stage II, cracks mainly propagated near the middle area. Cracks also appeared at the edge of the specimen until the load increase reached Stage III. The variation laws of the maximum crack width under the fatigue load are shown in Figure 18. The maximum crack width remained virtually constant at each stage; this width was controlled within the 0.05-mm maximum limit throughout the stage I and II, but not in the stage III.

The response shown by the number of cycles-strain curve (Figure 19) indicates that crack initiation does not occur in the rib-to-diaphragm joints, and no significant mechanical degradation occurred in the longitudinal specimen. When the loading cycles reached 6 000 000, the equivalent stress range of the rib-to-diaphragm joints exceeded the 71 MPa fatigue strength at 2 000 000 cycles specified in Eurocode 3 [30]. Because the equivalent stress range of the welded joint exceeds the fatigue strength, fatigue cracking was expected. The longitudinal fatigue performance of the steel–UHPC composite deck seems to mainly depend on the fatigue strength of the rib-to-diaphragm joints.

3.2.2. Mechanical Response. The displacements versus the number of cycles are shown in Figure 20. The local stiffness of the loading point varies with the number of actions, as shown in Figure 21. The local stiffness of the composite bridge deck below the loading point is defined as the ratio of applied loads causing the vertical displacement. In the first two stages, the local stiffness of the specimen remained virtually constant with the increase in the number of actions and degenerated during the loading process of Stage III. The
A reduction in longitudinal stiffness can be attributed to the UHPC cracking because it was the only damage observed during the fatigue test (Figure 21).

Distinct fatigue damage and mechanical degradation were not observed in the longitudinal specimen after 6,000,000 fatigue cycles under 1.5–3.0 times the designated vehicle loads. Therefore, the specimen has sufficient antifatigue capacity during its life expectancy and can withstand repeated traffic loads.

In the composite bridge deck, the stress concentration effect at the end of the weld at the rib-to-diaphragm joints is prominent; furthermore, the weld quality at the end of weld is difficult to ensure. Accordingly, this structural details with severe fatigue damage in the system are analyzed. The test results shown in the experiment indicate that the local strain of structural details is constant, and no fatigue crack is observed during the test. However, considering the essential attribute of large discreteness of fatigue strength of structural details, focus must remain on the fatigue performance of these details in an actual bridge structure.

3.3. Fatigue Strength. Based on the test results of longitudinal and transverse full-scale models, the composite deck structure system with large longitudinal ribs has satisfactory fatigue resistance under longitudinal and transverse fatigue loads. After 200,000 cycles of fatigue load as specified in the standard, 1.5 times the standard fatigue load was implemented; the structural mechanical properties remained stable. Although the UHPC layer cracked to some extent, the crack width was smaller than the nominal initial crack index (0.05 mm). In the accelerated failure test stage (Stage III), the longitudinal fatigue failure mode of the composite bridge deck structure with a large rib is manifested by UHPC layer cracking; the crack width exceeds the limit. In the transverse direction, the fatigue failure mode is indicated by UHPC layer cracking in which the crack width also exceeds the limit and weld root fatigue cracking at the details of the weld joint between the deck and longitudinal rib. In the full-scale longitudinal specimen test, although no fatigue cracking occurs at the rib-to-diaphragm joint, the stress level exceeds the fatigue limit in the accelerated failure test stage.

The nominal stress approach was applied to determine the fatigue strength of welded joints according to the measured fatigue strain range. Based on existing research, the fatigue failure criterion of rib-to-deck joints was defined as 25% of the drop in strain [31]. The points determined by the fatigue test results are located above the S–N curves [30], indicating that the fatigue performance of the welded joints fundamentally satisfies the design requirement (Figure 22).

4. Discussion

The test results show that the damage to the UHPC layer and head studs degrades the fatigue performance of the transverse specimen. The influence of damage on this performance can be determined by gaining understanding of the baseline mechanical properties of the transverse specimen when the UHPC layer and studs are not damaged through finite element (FE) analysis.
4.1. FE Model. To verify the accuracy of using the FE method for determining the stress field in the welded joints of the test specimens, numerical simulation of the transverse specimen was conducted using ANSYS software. Due to the symmetry of load and geometry, a semimodel is established to impose symmetric boundary conditions on the symmetric surface, as shown in Figure 23(a). The bottom of the FE model is fixed, which is consistent with the test specimen. The material constitutive laws of UHPC were identical to those in [32] and the adopted material parameters were consistent with the material properties in Section 2.2. The steel plate and rebars were simulated with elastic modulus of 200 GPa. The UHPC and steel plate were simulated using solid element (Solid45), and the steel rebars were simulated by truss element (Link8). The nodes of rebars were coupled with the UHPC layer. As shown in Figure 23(b), surface-to-surface contact element (Conta173 and Targe170) was used to model the normal and tangential interaction behavior between the UHPC and deck plate. The normal behavior was set as hard contact, and the tangential behavior was defined by a friction coefficient of 0.4. At the position of headed studs, two spring elements (Combin39) were used to simulate the transverse and longitudinal load slip behavior. The properties adopted for the headed studs were acquired from literature [33]. In vertical direction, the vertical displacement of the coincident nodes at the location of studs was coupled. A minimum refinement grid size of 0.5 mm is used to resolve the stress concentration problem near the welded joint.

4.2. Evolution of Fatigue Damage. The comparison between simulation and experimental results is shown in Figure 24. The simulation results are consistent with experimental data, and the error between the two is within 10%, indicating the effectiveness and accuracy of the FE method.

The fatigue failure criterion of the rib-to-deck joint was defined as 25% of the drop in strain, which can be calculated using equation (1):

$$\Delta \sigma_{eq} = \sqrt{\sum_{i=1}^{n} n_i (\Delta \sigma_i)^m} / N_{eq},$$

(1)

where $m$ represents the S–N curve parameters; $\Delta \sigma_i$ is the loading stress; $\Delta \sigma_{eq}$ is the equivalent fatigue strength; $N_{eq} = 2 \times 10^6$ cycles; and $n_i$ is the number of load cycles.

The accumulative damage with the number of cycles at the rib-to-deck joint can be calculated by equation (2):

$$D = \sum_{i=1}^{n} \frac{n_i}{N_i},$$

(2)

where $D$ is the damage index, and $N_i$ is the number of cycles in the S–N curves shown in Figure 24 for $\Delta \sigma_i$.

To determine the influence of the UHPC and studs of the composite bridge deck, another damage index can be calculated using equation (3):

$$D_{US} = D_a - D,$$

(3)

where $D_{US}$ is the damage index caused by the UHPC layer and headed stud; $D_a$ is the index obtained from the actual test; and $D$ is the damage calculated using equation (2) by the FE model.

The failure index of the UHPC layer and studs at the cracked rib-to-deck joint on the transverse specimen is shown in Figure 25. At the beginning of the fatigue test, there were almost no additional damage, then, tiny additional damage occurred after the initiation of cracks in the UHPC layer. At the loading stage II, the cumulative rate of additional damage increased to a certain
extent due to the several cracking of the UHPC layer. At the stage III, as the fatigue cracking of the headed stud, the additional damage increased rapidly. After $6 \times 10^6$ cycles of load, the additional damage value is approximately 0.53, which has a considerable influence on the fatigue performance of the composite bridge deck.

5. Conclusions

This study experimentally investigated the fatigue behavior of a steel–UHPC composite deck with large longitudinal ribs. The following conclusions are drawn.

(i) After 2,000,000 cycles of standard fatigue load and 1.5 times the standard fatigue load, no evident damage and mechanical degradation were observed in the transverse and longitudinal specimens. In addition, due to the excellent crack width control ability of UHPC, the test specimens satisfied the crack width limitation. Based on the foregoing, the proposed composite bridge deck satisfies the design requirements in terms of fatigue strength and durability.

(ii) The stiffness of the specimen distinctly degrades during the failure loading stage (3.0 times the standard fatigue load). The degradation mechanism of the transverse specimen involves the appearance of fatigue cracks at the longitudinal rib and weld of the steel bridge deck during the failure loading stage.

(iii) The effect of UHPC crack and headed stud fatigue failure on the damage accumulation of welded joints was quantified by FE simulation and experimental test; this influence accounted for 53% of the total damage.

Data Availability

The data used to support the findings of this study are available from the corresponding author upon request.

Conflicts of Interest

The authors declare that they have no conflicts of interest.

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