NUMERICAL MODELING OF THE STRESS RIBBON TERWILLEGAR PARK FOOTBRIDGE

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Abstract: The Terwillegar Park stress ribbon footbridge in Edmonton, Alberta began construction in 2014. The cable supported structures transfer large horizontal tension forces to the abutments, which are typically resisted by ground anchors. The vertical sag at midspan varies due to changes in temperature. Therefore, these structures do not require expansion joints, but geometry control during construction is critical. The superstructure analysis consisted of numerical modeling and analytical calculations. Results from both methods were compared to provide validation of the design. The numerical modeling methods are described. CSiBridge finite element software was used to create a staged construction analysis model. The construction model considered the various stages of construction including ground anchor stressing, tensioning the bearing cables, precast panel placement, cast-in-place concrete placement, post-tensioning the stressing cables, wearing surface and handrail installation. The model used constraints, as well as rigid and flexible links between nodes to accurately represent the appropriate construction stage. Construction and in-service temperature effects were included. Cable, truss, frame and shell elements were used for various bridge components. The numerical construction model compared well with the analytical results and will provide the designer with the ability to verify bridge geometry throughout all construction stages.

1 INTRODUCTION

Stress ribbon bridges consist of pre-stressing strands between abutments, which support relatively small prefabricated concrete panels that form the walking surface. These pre-stressing strands are referred to as bearing cables. The continuity of the structure is achieved by connecting the panels to form a ribbon, and post-tensioning the entire concrete ribbon with additional strands referred to as stressing cables.

Stress ribbon bridges do not have expansion joints or bearings, which reduces the long term maintenance costs. The thermal expansion and contraction is accommodated by the vertical “sag” of the superstructure, shown in Figure 1. As temperature increases, the superstructure moves downward, and the sag increases. As temperature decreases, the superstructure moves upward, and the sag decreases.

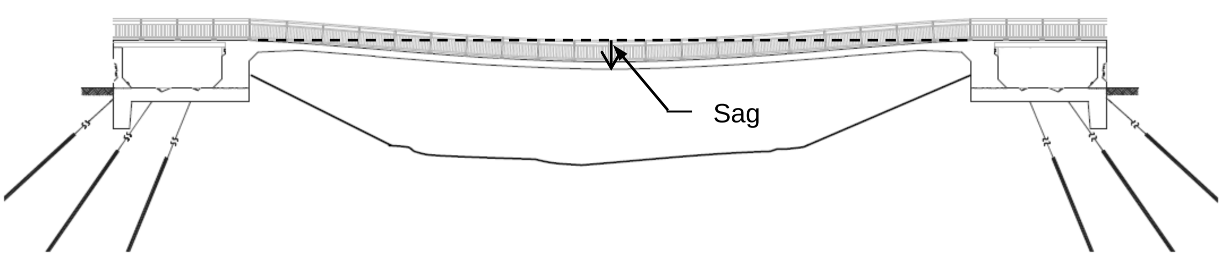

Figure 1: General layout of stress ribbon bridges (Stein and Birkle 2008)

It is important to monitor and control the vertical geometry of the superstructure during all stages of construction because the horizontal forces on the abutments changes with the sag. Figure 2 shows an idealized single-span, stress ribbon bridge of length, *L*, under a uniformly distributed load, *w*. Equation 1 explains how the horizontal tensile force, *H*, changes with sag, *Δ*.

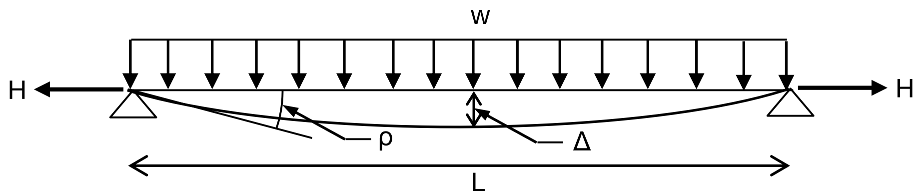

Figure 2: Analysis of a single cable subjected to a distributed vertical load, *w* (Stein and Birkle 2008)

[1] $H=\frac{wL^2}{8\Delta}$

The slope of the superstructure, *ρ*, is largest at the support locations. It is important to minimize the superstructure slope at the support locations so users of all mobility levels can access the bridge. However, this is difficult to achieve in design because the slope is dependent on the sag. As shown in Equation 1, as the sag decreases the horizontal tension force increases, creating larger demand on the foundations.

Stress ribbon bridges are slender and elegant structures that experience minimal vibration from pedestrian traffic. The construction method also minimizes environmental impact because no formwork is required for the superstructure (Stein and Birkle 2008). For these reasons, a three-span, stress ribbon bridge was chosen for the Terwillegar Park Footbridge over the North Saskatchewan River in Edmonton, Alberta. The nonlinear behaviour of cables and the importance of bridge geometry during construction create numerical modeling challenges that will be discussed specific to the project.

2 TERWILLEGAR PARK FOOTBRIDGE

The Terwillegar Park Footbridge is a 262 m long, three-span structure, with a 77-100-85 m span configuration. The 5.3 m wide concrete deck is only 265 mm deep making the span length to superstructure depth ratio over 370 for the 100 m middle span. Figure 3 shows the general layout of the bridge.

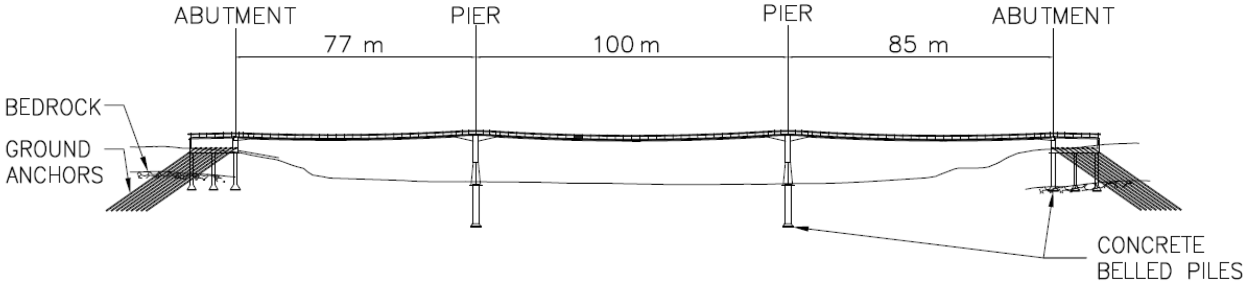

Figure 3: General layout of Terwillegar Park Footbridge in elevation

The concrete abutments and piers are supported by concrete belled piles drilled into bedrock. High strength threaded bar ground anchors are grouted 15 m into the bedrock layer. The large horizontal loads from the superstructure are resisted by 77 tensioned ground anchors at each abutment. Cast-in-place concrete closure pours are used at the abutments and piers to connect the deck panels to the substructure. The deck consists of 86-2.64 m long precast concrete panels which are supported by the bearing cables. Erection pins are installed during panel placement to allow the panels to be hung from the bearing cables. The precast panels contain post-tensioning ducts in longitudinal troughs. The deck is made composite by casting both the longitudinal trough and panel joints. Figure 4 shows a cross section of a typical precast panel segment.

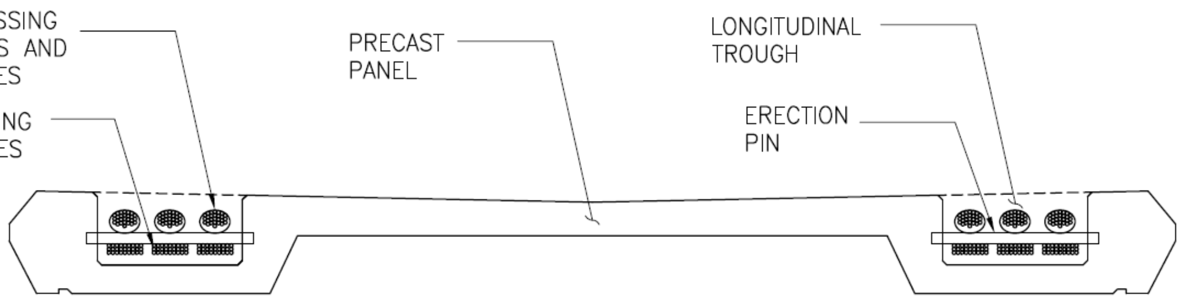

Figure 4: Cross-section of a typical precast deck panel

The bearing cables are supported on smooth saddles at the abutments and piers that allow the cables to slide over the supports as the precast panels are placed. Flat observation areas are provided at the piers. Due to the inherent increase in deck slope at support locations for stress ribbon bridges, the flat observation areas were made by using two saddles supported on beams protruding from the pier centrelines located 4.45 m from the pier centrelines, as shown in Figure 5.

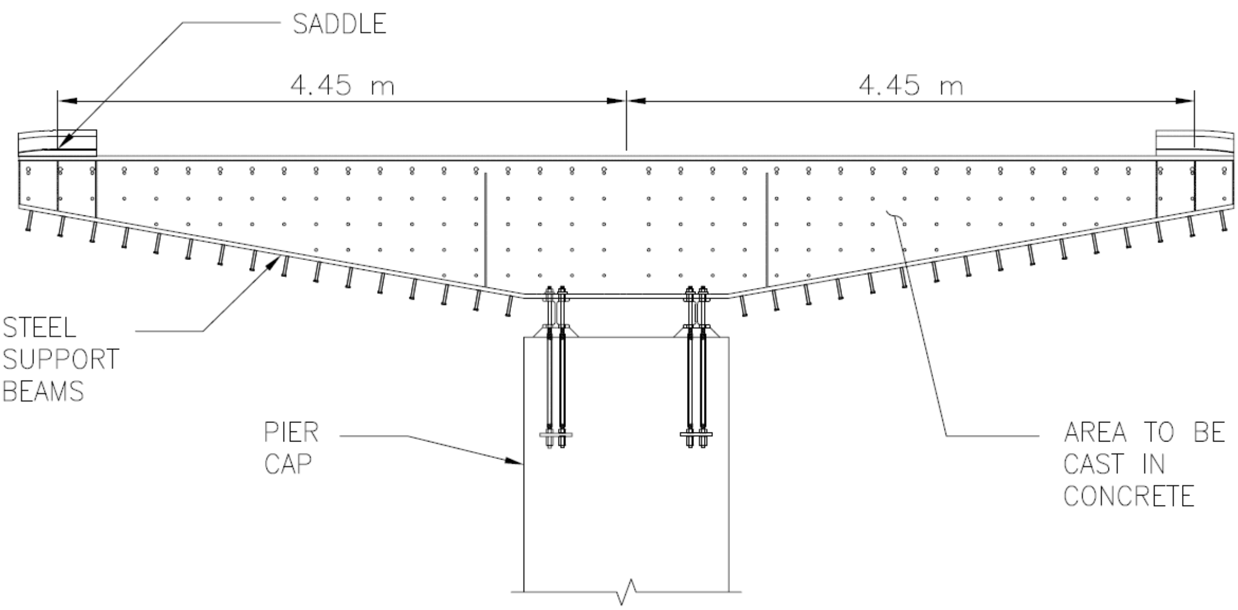

Figure 5: Pier saddle locations to create flat observation platform

2.1 Construction Stages

The construction sequence is important for bridge geometry and structural capacity. The expected construction stages are described.

Once the piers and abutments are constructed, 13 of the 77 ground anchors at each abutment are stressed. The bearing cables are then installed and stressed. After stressing the bearing cables, 16 more ground anchors at each abutment are stressed. The precast panels are then placed on the bearing cables in each span. To minimize the amount the bearing cables slide over the saddles, the panel placement procedure should minimize unbalanced loading between spans. After all of the panels are placed, the stressing ducts and cables are placed in the longitudinal troughs. Then the longitudinal troughs, transverse panel joints, and closure segments at the piers and abutments are poured. Once the cast-in-place concrete has reached sufficient strength, the stressing cables are post-tensioned and the remaining 48 ground anchors are stressed. The stressing cable ducts are then grouted, once it hardens the entire system becomes composite. Finally, the wearing surface and handrail are installed.

The reason for staging the ground anchor stressing is to reduce the demand on the concrete piles. Stressing the ground anchors applies compression to the concrete piles. Stressing the bearing and stressing cables applies tension to the concrete piles toward the superstructure. By staging the stressing of the ground anchors, the demand on the concrete abutment piles is reduced because the compression and tension are better balanced throughout construction.

3 CONSTRUCTION MODELING

Due to the nonlinear behaviour and importance of analyzing the structure throughout all stages of construction, CSiBridge finite element analysis software was used (CSiBridge 2014). The modeling methods chosen required structural elements to be added during different stages of construction, which is a feature of CSiBridge. A 3-dimensional model was not required because consideration for torsional effects or lateral loads was not the purpose of this analysis. Therefore, a line model was used to analyze the entire bridge structure.

3.1 Element Definition

The bearing cables were modeled using cable elements. Defining cable elements requires more information than the geometry and material properties because the stiffness of cable elements depend on the tension force and profile of the cable. The bearing cable elements were defined using the self-weight of the cable and an axial tension force provided from the analytical results. After analyzing the model with only the bearing cable, the cable sag was compared with the analytical solution. This provided confidence with the initialized model consisting of only the bearing cable.

The precast deck panels were modeled using frame elements that were the same length of each panel. During panel placement the moments were released at the ends. This ensured that moments were not transferred between panels prior to casting the transverse joints. These end releases were removed once the panels became composite.

The stressing cables were modeled using truss elements. The initial geometry of the stressing cables is defined by the panel troughs they are placed in. This avoided the requirement to define cable geometry at an intermediate step in the staged analysis. Because truss elements resist only axial loads and the stressing cables are only subjected to tension, truss elements were a suitable substitute for cable elements for this design. Using truss elements for the stressing cables greatly reduced the model complexity. An internal tensile axial strain in the truss elements were introduced as a load. This strain was calculated to be equivalent to strain expected from the applied post-tensioning force.

The ground anchors were also defined using truss elements connected to a pin support in the bedrock an abutment node at the top. Truss elements were appropriate because the ground anchors never experience compressive stresses. The concrete abutment and pier piles were modeled as frame elements with vertical roller supports, to represent soil bearing, at the pile tips. Frame elements were used because the large diameter concrete piles were expected to resist longitudinal and transverse bending moments. Horizontal soil springs were used along the entire pile length to represent soil restraint. The spring constants were based on recommendations in the geotechnical report.

Analysis of the substructure is important, however, it is not the focus of this construction model. The substructure components are briefly described. The abutments were modeled with thick shell elements. The stressing and bearing cable forces are transferred through thick rib walls to the abutment slab, and then to the ground anchors and piles. The pier shafts and observation area support beams were also modeled using frame elements.

3.2 Staged Analysis

A staged analysis model was created to follow the construction sequence. The construction sequence was broken into the following stages for modeling purposes:

1) Substructure construction
2) Stress 13 of 77 ground anchors in each abutment
3) Install and stress bearing cables
4) Stress 16 of 77 ground anchors in each abutment
5) Place precast panels
6) Install stressing cables and ducts; cast longitudinal troughs, transverse joints, and closure pours
7) Cast-in-place concrete achieves sufficient strength and system becomes composite
8) Post-tension stressing cables
9) Stress remaining 48 of 77 ground anchors in each abutment
10) Grout stressing cable ducts
11) Grout hardens and stressing cables become composite with structure
12) Apply the dead load of the wearing surface and handrail

Each stage started from the conclusion of the previous stage. This meant the system displacements, stresses and stiffness were carried forward and the new loads, boundary conditions or elements were introduced in the following step. The staged construction analysis provided several challenges that required innovative solutions.

Each bearing cable, precast panel, and stressing cable element was defined between two nodes that were unique to that specific element. The lengths of all the individual bearing and stressing cable elements were the same as the precast panels. This allowed for the bearing cable, precast panel and stressing cable nodes to be at the same location. When nodes are at the same location in a coordinate system they are defined as coincident nodes. The vertical displacement between these coincident nodes was constrained. This meant that the relative vertical displacement between the coincident nodes was restrained for the entire staged analysis. Figure 6 shows the elements and coincident nodes. For clarity, Figure 6 shows the elements and nodes separated by a small vertical distance, but in the actual model, the nodes and elements are at the same elevation.

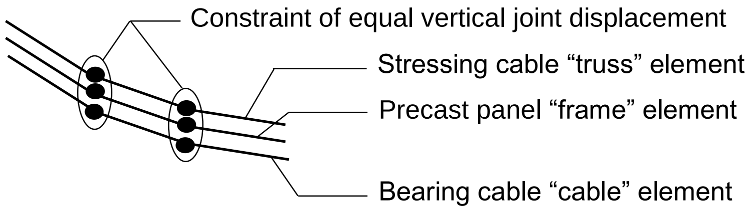

Figure 6: Vertical constraint between coincident nodes of cable, frame, and truss nodes

By constraining the vertical displacement, the nodes could only displace horizontally relative to each other, as would be expected while placing the precast panels or post-tensioning the stressing cables. The vertical constraint also allowed for the bearing cables to slide freely across the saddles located on the pier observation beams and at the abutments.

The bearing cables were installed and stressed in stage three of the analysis but the precast panels and stressing cables were not added until stages 5 and 6 respectively. Because their nodes were added when the model was initialized, a “flexible” link was required to force the stressing cable and precast panels to follow the horizontal displacement of the bearing cable so their elements could be added at the correct deck profile. The flexible link had a very small stiffness in the global horizontal direction. This allowed for the panel and stressing cable nodes to follow the bearing cable nodes when they were not connected to elements. When the panel and stressing cable elements were added to the model the links were flexible enough to have no effect on the relative horizontal displacement between bearing cable nodes and both the panel and stressing cable nodes. This displacement condition would be expected prior to the systems becoming composite. Figure 7 shows the flexible links which restrict displacement between the attached nodes in the global horizontal direction. The precast panel and stressing cable elements are excluded from Figure 7 because at this stage in the analysis they have not been introduced to the model. For clarity, the nodes in Figure 7 are vertically spaced apart vertically to show the flexible links. However in the construction model the nodes were at the same elevation.

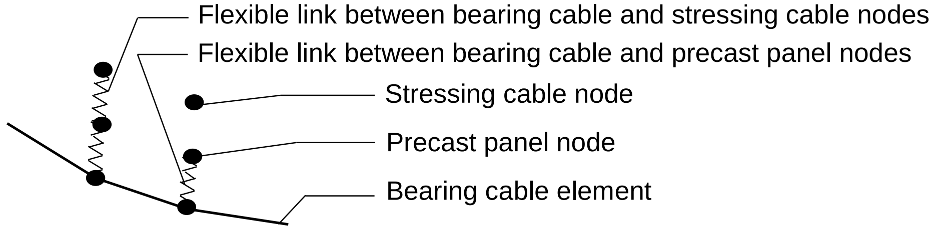

Figure 7: Flexible links in the global horizontal direction

The self-weight of the precast panels was accounted for with a unit uniformly distributed load applied to each panel. This allowed the for dead load to be applied and scaled as needed depending on the number of panels placed and then multiplying by the weight of the panels, stressing cables and wet concrete. The wearing surface and handrail loads could be applied in this manner as well.

Once the troughs and joints were cast and hardened, the end moment releases on the precast panel elements were removed. This allowed for bending moments to be transferred from panel to panel. In addition to this, “rigid” links were introduced between the bearing cable and panel nodes only. No relative displacement between bearing cable and precast panel, in any direction, was permitted. The relative vertical displacement was restricted due to the constraint, and horizontal displacement, due to the rigid link. This was done to simulate the composite behaviour between the bearing cables and precast panels. The stressing cable nodes were still free to displace horizontally relative to the bearing cable and precast panel nodes until stage 10. Figure 8 shows the rigid link between the bearing cable and precast panel nodes.

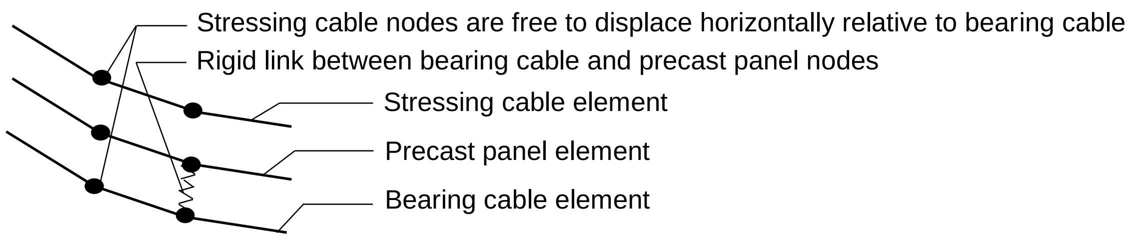

Figure 8: Rigid links in the global horizontal direction

After the concrete has hardened, the stressing cables are post-tensioned. Because these elements were actually truss members, the stressing force was not applied in the same manner as for the bearing cables. A tensile strain was applied to the stressing cable elements that would produce the same stress in the elements as the post-tensioning force. After the post-tensioning, the load of the wet grout in the cable ducts was added and, once the grout achieved sufficient strength, the stressing cable becomes composite with the entire bridge system. To reflect this in the model, rigid links were added between the stressing cable nodes and the bearing cable nodes, making the entire system composite.

Finally, the weight of the wearing surface and handrails were added.

A tensile strain was applied to the ground anchor truss elements for each stage the ground anchors were stressed. The staging of these ground anchors also identified how the stresses in each row of ground anchors vary at the end of construction. Anchor rows where the first anchors were stressed have lower stresses than the final rows. This is because as the final rows are stressed, the strain in the first rows is reduced because the abutment is displaced downward.

4 MODELING ISSUES

Two-joint, zero length links were used for both the “flexible” and “rigid” links. When the links were defined, the length between the coincident nodes was zero. The flexible links were introduced to the model prior to any load steps, meaning the distance between the nodes they were linking was still zero. The rigid links were introduced after various load steps, and relative horizontal displacement between the coincident nodes had occurred. This meant that the length between nodes was nonzero. Through various model trials on simple models, this was observed to not be an issue. In the simple models, there was no relative displacement between nodes joined by a zero-length rigid link. This provided the designers with confidence that the zero-length rigid links were suitable for this application.

Applying the post-tensioning force as a strain was an iterative approach. As expected, when a tensile strain was applied to the stressing cable truss elements, the superstructure displaced upward. This upward movement meant that the strain in the stressing cable was lower than the expected post-tensioning force. In order achieve the desired final post-tensioning stress in the cable, the strain had to be increased above the theoretical value. This was done for this analysis, but it created some questions from the designers. When the stressing cables are post-tensioned onsite, will the jacking force be measured by the cable elongation, strain, or pressure gauge, stress? If it is measured by elongation then the jacking force will be lower than expected because some of the elongation is not due to the strain on the cable, but rather the upward movement of the superstructure. These questions will be considered when reviewing the contractor’s erection procedures.

Another interesting observation during post-tensioning of the stressing cables is that the system has similarities to an upside-down arch. Because the abutments are relatively fixed, the post-tensioning applies a uniformly distributed load upward on the concrete panels. This creates axial compression in the panel elements, similar to that of an upside-down arch.

5 COMPARISON WITH THEORETICAL MODEL

The numerical modeling techniques have been described. The numerical model was validated by comparing it with analytical calculations. A comparison between the analytical and the numerical analysis results are shown in Tables 1 and 2 for various construction stages.

Table 1: Horizontal tension force, H, at midspan of 100 m span

Analysis Stage	#	Analytical Analysis (A) (kN)	Numerical Analysis (B) (kN)	B/A
Bearing cable stressed	3	18347	18835	1.03
Stress 16 ground anchors	4	18616	19065	1.02
Place precast panels	5	21776	21017	0.97
Post-tension stressing cables	8	28619	29390	1.03

Table 2: Vertical sag, Δ, at midspan of 100 m span

Analysis Stage	#	Analytical Analysis (A) (mm)	Numerical Analysis (B) (mm)	B/A
Bearing cable stressed	3	98	94	0.96
Stress 16 ground anchors	4	96	93	0.97
Place precast panels	5	1372	1438	1.05
Post-tension stressing cables	8	1366	1424	1.04

As shown in the tables above, the analytical and numerical analyses correlate well, within 5% for all stages investigated. Although not all analysis stages were compared, based on these results, the intermediate stages investigated in the numerical model are expected to be reliable.

6 CONCLUSION

The excellent correlation between the numerical results described here and the analytical results provide the designers with confidence with the numerical construction model results. This model will be a valuable tool to monitor the bridge geometry as construction progresses. The construction temperature and live load effects can easily be incorporated to the model as needed. It also will help troubleshoot onsite problems efficiently. In addition to aiding geometry control as construction progresses, creating this detailed model has helped the designers predict and mitigate potential issues that otherwise would not have been thought of.

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8 REFERENCES

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