STEEL DECKS FOR RAIL BRIDGES

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ABSTRACT
Durability of bridge decks is of vital importance to the Railroad industries. Replacement of deteriorated decks represents a major maintenance item, disrupts operations, and is a routine capital investment. Steel decks, when properly constructed, have proven to be a more durable option than other alternates. AREMA Committee 15 for Steel Structures is currently evaluating the necessary design, fabrication, and construction parameters. Parallel to that effort, a steel deck is under development for use with standard steel rail spans. The decks will be fabricated from plate material and common steel shapes in order to provide a long term, cost effective decking solution. The overall goal is to create a deck system that is equally as durable as the structures they are supported by and can be installed by traditional means, following a new AREMA provision.

BACKGROUND
On average, timber decks for Rail bridges are replaced on intervals of 15 to 25 years. Similarly, the expected life of a standard highway concrete deck is 20 to 30 years. The expected design life of a steel orthotropic deck has been equal to the design life of the structure itself, historically, 75 years. Modern expectations for steel decks are for over a 100 year design life.

In contrast to the expected life, however, when not properly installed or fabricated, steel orthotropic decks have a history of fatigue and hot-cracking related problems. This is due to the complex nature of the structural demands on the orthotropic system. Rarely, however, is strength of the orthotropic deck a controlling consideration. A modular steel deck combines the best of both orthotropic decks and modular decks in that it reduces the complexity of the system analysis, yet maintains a modular approach to construction.

Considering the analysis, similar to a one way slab design, the proposed steel deck configuration places the main load resistance transverse to the stringers. The basic geometry is shown as a sketch in Figure 1.

The idea of a flat plate steel deck is not new. Steel decks are currently used in rail construction, primarily for through plate girders (TPG) with continuous floorbeams (Figure 2). They are also found on deck plate girders (Figure 3). While TPG configurations with floorbeams are considered a standard practice, the use of steel decks with more common deck plate girders is much less common. As can be seen in Figure 3, the required beam spacing is relatively close
together and additional supporting brackets are required to support the overhang and walkway.

Figure 1. Sketch of steel deck on stringers, deck ribs (orange) are transverse to the stringers (red)

Figure 2. Through plate girder, typical steel deck with floorbeams spaced at 2ft

Figure 3. Deck Girders with steel deck and support brackets

Alternate configurations with wider beam spacing require the deck to be transversely supported between the beams.
The initial concept that spurred a steel deck with transverse stiffeners was to eliminate the extra detailing required for the support brackets while still providing the durability of steel. The proposed deck with transverse ribs would eliminate several support details and facilitate installation.

**LOADS**

A railcar is dependent on the longitudinal placement of the load. The AREMA design load is the Cooper E-80 load consist and alternate load (Figure 4) (AREMA 2013).

Train loads are supported by ballasted or open deck bridges. In open decks, the tie bears directly on a structural supporting member, most commonly with Timber or FRP ties that bear directly on stringers or deck girders (Figure 5). In a ballasted deck, ties are supported by ballast, which in turn distributes the load to a supporting deck.

For this steel deck under development, railroad loading with a ballasted deck is investigated. As mentioned, the rail car loads are distributed through the ballast and this creates load pressure bulbs below the ties as shown in Figure 6 (Hay 1982).

For strength modeling purposes, the controlling axle load is the 100-kip axle load spaced at 5ft. Thus, following the procedures for distribution of wheel loads in accordance with Hay, it can also be assumed as an equivalent point load located under each tie, as distributed through the ballast. Also, since the exact location of a tie can not be predetermined, the loads are applied at the tie spacing and moved longitudinally along the deck in order to capture the worst case loading condition (Figure 7).
Fatigue design used the fatigue limits calibrated for rail loads in AREMA (2013). A refined fatigue evaluation was also done in accordance with AREMA 7.3.3.2, using actual expected rail car loading to predict the expected life of the structure. To do this, the operational planning for rail car weights needs to be applied. Common car consists (2013) include 315k, 286k, and 263k cars, with axle loads of 78.75k, 71.5k, and 68.75k, respectively. A similar exercise was done for these axle loads.
CURRENT AREMA REQUIREMENTS THAT APPLY TO STEEL DECK DESIGN
Design guidance for steel decks is currently limited to information in AREMA Chapter 15 in the form of allowable stresses (AREMA Chapter 15, Table 15-1-11), and the applied load noted in the previous section. The obvious limiting stresses are:

- 0.55 $F_y$ for tension in the extreme fibers of built up sections subject to bending (net section)
- Appropriate limits for compression in extreme fibers of flexural members of welded built up sections
- 0.35 $F_y$ for shear stress in rolled beams or plate girders (gross section)
- Appropriate limits for axial compression of centrally loaded members
- Appropriate Fatigue Category limit
- Displacement requirements of AREMA 15-1.2.5

Each of these limits was used to design for particular portions of the steel deck. Unfortunately, several design conditions are not directly addressed by AREMA. This will be discussed more in the sections for strength and fatigue design.

GEOMETRY
The basic layout is shown in Figure 1. Figure 8 shows a sketch of the geometry used for the development design of the steel deck. The critical dimension is the stringer (or deck girder) spacing. This is effectively the unsupported length of the deck for live load. For the design project, 9ft center-to-center stringer dimensions controlled the design.

Limited attention was given to span length. It is acknowledged that deformation of the main supporting members under load will induce compressive strain at the top flange for positive moment. However, the attachment configuration was configured such that the deck is allowed to expand and contract in the longitudinal direction. This will be discussed further in the detailing section.

STRENGTH DESIGN
For the deck girder system, the Cooper E80 loads induce various primary stress conditions that need to be evaluated. The following basic requirements were used for the preliminary design:

- Transverse bending of the panel between the stringers
- Longitudinal bending of the deck plate

Emphasis of this section will be given to the calculations that lead to the selection of the supporting shape for the deck to resist the maximum positive moment between the supporting stringers. The other load conditions are checked after sizing the member and then the shape is adjusted as necessary.

Required bending resistance of the deck is very dependent upon the girder spacing and the assumption of how the deck is supported. That is, the majority of the vertical resistance is provided by the web of the supporting stringer, but the flange provides additional resistance depending on the thickness of the components and the addition of stiffeners (if any) to the
stringer section. For the design, it is assumed that the deck is simply supported between points of vertical support, the webs.

Additionally during design, support effect of the flanges was not considered. This was done to simplify initial design considerations and also for ease of modeling. Also, consideration was given to the fact that the results would be slightly conservative for positive bending. Moreover, the buckling and shear of the vertical deck members would be able to be better scrutinized for the resulting concentrated load. This would also account for out-of-flatness fabrication tolerances in the flanges which could lead to the deck not being supported at the flange edges.

The preliminary design focused on sizing of the supporting shape. As such, a basic deck plate thickness was assumed based on the unsupported length and the bending demand per foot of deck was determined. The plate was initially sized to the anticipated location of highest demand of the deck, at the end of the deck; continuous on one end, and simply supported on the other. This design is one place where the application of load and the resulting stresses are not clearly indicated in AREMA. Four our design, due to the ratio of the tie width to the expected shape spacing, an even distribution of load was assumed under 1/3 the length of the tie.

For the shape selection, based on the loading demands of the Cooper E80 and alternate live load distributed through the ballast, the bending moment demand in the transverse direction is 31-k-ft per foot. Thus, the required section modulus is 13.5-in$^3$ per foot (assuming a material strength of grade 50 steel) for strength. In accordance with AREMA deflection requirements, deflection is being limited on the deck to L/640, where L is the girder spacing of 9-ft. The corresponding required moment of inertia is therefore 84.5in$^4$ per foot.

**SHAPE OPTIONS**

There are options as to how the strength and stiffness demands can be resisted. Several shapes were evaluated for both material and fabrication efficiency. This includes:

- W-Shape
- T-Shape
- Structural tube
- Channel section
- A combination of shapes

**W-Shape**

Shown in Figure 9, the W-shape has the advantage of being strong, compact and an efficient section on its own. It also allows for a slightly greater spacing of the sections as the top flanges offer support to the deck plate, shortening the effective span of the plate. When considering the composite action of the steel deck plate, however, the top flange becomes superfluous, or at least inefficient. This naturally leads to evaluating a T-shape.

**T-Shape**

The advantage of the T-shape is that, similar to an orthotropic deck, the deck plate becomes the top flange and offers good section efficiency (Figure 10). There are disadvantages to using this shape. First, the span of the plate is from stem to stem, decreasing the possible spacing of the
shapes. Second, access for placing the longitudinal weld can limit section selection to shorter shapes with narrow flanges.

**Structural Tube**
The next shape considered is the structural tube, rectangular or square (Figure 11). The advantage of this shape is the additional torsional stability and an increased spacing with respect to the t-shape. The disadvantages are having an inefficient top flange, having a constant thickness that leads to inefficiencies, and having rounded edges which adds a level of weld complexity when attaching it to the deck plate.

One additional configuration that was considered was butted structural tubes; the top flange forms the deck surface. This turned out to be a less efficient design due to the number of sections required for each panel. It also placed the top butt weld of the two rounded sections in direct tension as a Category F fatigue detail.

**Channels**
Channels offer another option (Figure 12) but have the disadvantage of being asymmetric on their vertical axes and offer less torsional stability than other options. Where channels were found to be useful, however, was at the ends of panels. They are able to minimize the unsupported edge distance.

**Combination**
The goal of developing this plate and shape configuration is that engineers and fabricators need not be limited by a single shape per deck or per deck panel. As each situation presents a different set of demands, one can imagine selecting any particular shape to meet the needs with the fabrication details already established.
Selection for the proposed panel
A combination of T-shapes and channels was eventually selected (Figure 7). The T-shapes are
the primary shape across the panel. But, the flange of the T-shape led to larger unsupported
lengths at the end of the panels. To counter this, a channel is placed at each panel with the
channel web facing the edge end and placed relatively close to the edge of the plate.

The selected shapes are WT8x15.5 and C8x18.75 creating a transverse bending section modulus
per foot of 23.2in³. The moment of inertia is 134in⁴. Thus, the demands are met for the primary
strength design.

REFINED ANALYSIS
At this stage, the initial parameters are set for the design: plate size; shape size and spacing. To
fully evaluate the design and to establish that the decks will be satisfactory, the evaluation
needed to go both address limit states set by AREMA as well as additional requirements.

Shear
Shear of the deck is a potential failure mode in a couple locations. One is the shear stress in the
weld between the shape and the deck plate. AREMA and AISC provide very clear guidance on
the horizontal shear continuity of the weld. The other is the shear of the deck at the supports.
For this, similar to a rolled shape or built-up section, we took the cross sectional area of the
shape’s web as the mechanism of resistance at the point of support. This was deemed to be a
good conservative value for a non-controlling condition. Shear stresses were compared between
basic hand calculations and the finite element model, and were found to be similar.

Buckling of the deck plate
The deck plate is essentially a thin plate supported at regular longitudinal intervals by the deck
shapes. From one perspective, the deck plate can be thought of as the compression flange of a
built-up member. From another, it is a compression member (from bending) that is also being
subjected to an out-of-plane pressure on one face (from ballast pressure). To add to the
complexity, it is potentially receiving multi-axial compression as the beam or girder that supports
it bends. To breakdown the problem into more manageable pieces, we borrowed from the
FHWA Orthotropic deck design manual (2012) and evaluated two design conditions.

First, the deck plate was designed (per foot) for the bending condition of the deck, where the
plate functions as the compression flange of the section. Compression properties are based on th
radius of gyration derived from the plate thickness and tributary width. In calculating the
resistance, the plate could be considered continuously braced. However, to err towards
conservatism, the effective transverse bracing (compression flange brace spacing) was set at the
longitudinal shape spacing (Timoshenko, 1961).

Second, the deck plate was evaluated for local stability as a simply supported plate with edge
restraint. The loading for this condition is axial compression along two edges with an evenly
distributed pressure on one surface. These simplified plate dimensions were evaluated for a
length to width ratio of 1:1 and 2:1. This evaluation was done with a non-linear finite elements analysis.

**Buckling or crushing of the supporting shape vertical member**
The web(s) or stem of the supporting shapes must also not buckle or crush where the deck sits on the supporting members. AREMA 15-1.7.7 sets the allowable stress and guidance for what material should be considered for compression at a point of bearing. In particular, it recommends limiting the compression area of the flange to the edge distance or 12 times the web thickness. However, based on the support being a beam or girder web instead of a bearing plate, we also introduced a limiting width of the smaller of four times the web thickness or one quarter of the flange width (Figure 13). Of note is that this reduction did eliminate some of the more slender T stems from consideration.

**FATIGUE DESIGN**
Of particular importance, especially for rail decks, is fatigue. Each passing train can impart hundreds if not thousands of stress cycles. Fatigue design is done with the same axle loads as are used for strength design, the Cooper E-80 and alternate live loads. Consideration can also be given to rail operational loads, and the load frequency imposed by rail consists of heavy axle cars. The known operating limits for the span being evaluated is a 315-kip, 4 axle car consist (78.75k per axle).

For fatigue, there are three conditions that are of particular importance. First, there is the fatigue induced by primary bending of the section. For the proposed panel in bending, the two details of concern are the flange of the shape in tension (plain steel, Cat. A) and the shape-to-deck plate weld (longitudinal fillet weld, Cat. B). Second, there is the longitudinal bending of the plate resisted by the supporting shape. This detail takes guidance from the FHWA Orthotropic Manual (2012) that considers the fillet welded connection between the section and the deck plate at Category C for the local structural stresses induced by bending at the weld toe. Third, there is the negative bending fatigue over the supporting structure which places the longitudinal fillet weld (Cat. B) in tension.
Each condition was evaluated for the E-80 axle loads and various rail car consists, but in particular for the heavy rail 315-kip load car.

A sample of the stress profile is shown in Figure 14. This figure shows the bending about the y-axis (about the longitudinal axis, i.e. plate bending over the supporting T-shape web) on the plate. As expected, the longitudinal bending is highly localized to the applied loads. Similar checks are done for the other fatigue loading conditions.

It should be noted that for each case, except for the negative moment over the support, the applied forces place the welded connection in compression.

For this panel design, there is effectively no negative moment in the deck at the support. For the stringers spaced at 6ft, and other configurations, more attention will be spent evaluating the negative moment regions.

![Figure 14. Deck Panel Local Stress Analysis showing bending of the plate in the longitudinal direction](image)

**ADDITIONAL DETAILS**

The basic design has been established for strength and stability. For detailing, several considerations were made with a goal to ease cost fabrication complexity. For example, the proposed welding procedures were limited to fillet welds to minimize cost. (Full penetration groove welds are used to splice the deck plates as necessary.)

The various proposed details are shown in Figure 15. With a deck expected to have a service life to match the superstructure, it is important to insure that the connections be equally durable. As such, combination of both hook bolts and thru bolts was used to attach the decks to the girders. The bolts were designed and spaced to resist longitudinal forces that could develop in the panel, and although not specifically prescribed for deck design, the stability check prescribed in AREMA 15-1.3.10 was also checked.

**Curbs**

For the curb, there are options. One is to use a structural tube. This would provide a measure of edge restraint but can make the connection to the deck panel more challenging. Another option is to use a channel section, which also provides edge restraint to the deck, but not as much in
torsion. However, a solid weld or other connection to the deck also needs to take into consideration the drainage needs for the structure.

Inter-panel continuity
One advantage of a rail deck is that maintaining a smooth riding surface is not a mandate. Thus, a simple solution is used to bridge between adjacent panels and serves as an expansion point as well. A T-shape of sufficient flange width is slotted between panels. The stem keeps it in place, and the flange bridges the gap and need not be connected to the panels (Figure 16).

FABRICATION
Ease of fabrication is also important to make these decks a viable option. Experience with orthotropic deck construction has demonstrated that welding of ribs to plate can become overly complex. However, these modular panels are much less complex in their detailing. Fillet welds on each side of the shape is the primary weld, thus avoiding any complex or more expensive types of welds. Pre-bending is still a concern, but once a fabricator gains experience with a particular deck, the process will be repeatable.

Additionally, fabricators were contacted to evaluate which type, size, and spacing of shapes would make the welds feasible. With potentially close spacing of the shapes it is important to provide sufficient access for automated welding. T-shapes with particularly short webs and wider flanges were eliminated for this first design due to access; any other members that were tall and required tighter spacing also were ruled out. Eventually, these limitations might be eliminated with an automated welding traveler, should these decks ever be mass-produced.
Advantages
The primary goal of introducing this steel deck is durability. Several experts have indicated that the steel deck is the only deck with a potential to match the service life of the superstructure (FHWA, 2012).

Another consideration is expedience of placement. As with other panelized systems, the steel deck panel can be placed in a modular fashion.

Unlike concrete decks, but similar to timber decks, the steel deck is also relatively lightweight. One of the goals of the initial study was to design a panel of similar weight as a timber deck for the rail at the same weight. As such, the panels can be installed without any heavy crane equipment, which greatly increases the versatility of where they are able to be installed. For example, using the same configuration of 9ft girders spacing, the required timber deck would be a continuous 9in deep timber with an installation weight of approximately 780-plf. Comparatively, the proposed steel deck with a WT8x13 and 5/8in steel plate would weigh 720-plf. Thus, the same lifting equipment that is currently being used for timber deck installation could still be used. A comparable deck using a TS8x3x3/8 would weigh 950-plf.

A further goal was to eliminate additional details and field assembly details. As mentioned previously, the transverse stiffness of this system eliminates support bracket details. It also provides a flat and safer working environment as soon as the panel is set in place.

Cost
The cost of the panels is not insignificant, especially in comparison to a standard rail timber deck. The same deck comparison that was used for the weight above is used again here to compare cost. All estimated costs are as of the date this article is written. The estimated cost of timber is $1750 per thousand-board-foot. This equates to $171 per foot of deck. A concrete deck panel assuming $1 per pound of reinforcing and $1500 per cubic yard of concrete equates to approximately $1140 per foot of deck. For the steel deck panel under development with WT8x13 and a 5/8in deck plate, assuming a fabricated cost of $2.00 per pound, would be $1450 per foot of deck.

Other life-cycle cost reductions, however, also offset the upfront increase in the cost of the steel deck. For the common deck beams (Figure 3), the removal of the brackets and any additional false work also offsets the increased cost of the deck panel. But, perhaps more importantly, operational costs associated with deck replacement are potentially eliminated for the life of the structure.
REFERENCES

AASHTO LRFD Bridge Design specifications (2012), American Association of State and highway Transportation Officials, Washington DC


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Overview

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- Additional Details
- Remarks

Background

- Motivation
  - Timber decks: 15-25yr life
  - Concrete decks: 20-30yr life

- Inspiration:
  - Orthotropic Decks $\rightarrow$ Durability
  - Precast concrete decks $\rightarrow$ Modular

Background

- Decks are a major recurring capital investment
- Goal:
  Develop a durable and adaptable deck system for railways with the design life intended to match the supporting structure

Typical Decks

- Previous use of steel decks
  - TPG:
  - Deck beams:

- Common 2 and 4 Beam/Girder bridges
  - Structural decks that distribute loads transversely
Typical Decks

• Can we provide a modular steel deck solution?

Typical Decks

• Meet the goals:
  – Fabricate from common plates and shapes
  – Cost effective
  – Ease of installation (ABC, Rail)
  – Modular system
  – Long Life
  – Multiple Applications (Existing and New)

Loads and Loading

• Cooper E-80 and Alternate:
  – 100-kip axles control for localized design

Loads and Loading

• Load Path
  – Rail is supported by tie
  – Tie is supported by ballast which transfers load to the deck (ballast deck)

Loads and Loading

• Analysis
  – Concentrate on Ballasted Deck
  – Ballast Deck Applied Loads (pressure bulbs)

Loads and Loading

• Visualization of load path for 2-girder ballasted deck
  Effectively a local pressure load on deck
**Loads and Loading**

- Initial modeling: pressure bulbs as point loads:

**AREMA**

- Chapter 15, Steel Structures, Table 15-1-11
  - 0.55 $F_y$ for tension in the extreme fibers of built up sections subject to bending (net section)
  - Appropriate limits for compression
  - 0.35 $F_y$ for shear stress in rolled beams or plate girders (gross section)
  - Appropriate limits for axial compression
  - Appropriate Fatigue Category limit(s)
- Displacement requirements of AREMA 15-1.2.5

**AREMA**

- Additional design checks
  - Items not covered in Table 15-1-11
  - Additional Fatigue considerations
    - (covered later)

**Geometry**

- Critical dimension is stringer spacing
  - Effective unsupported length
  - Varies from 3ft to 9ft
- Transverse rib spacing
- Deck height restrictions

**Strength Design**

- Evaluation of primary stresses
  - Transverse bending
- Evaluation of other stresses
  - Deck plate bending (Lat. And Long.)
  - Buckling of deck plate
  - Buckling of supporting shape’s vertical member (e.g. web plate)
  - Shear of deck panel
  - Etc.

**Strength Design**

- Basic assumptions:
  - Ignoring girder flange support
  - Loading:
    1. Treat pressure bulb loads as point loads
    2. Treat pressure bulbs as pressure
    3. Vertical + Breaking/Traction
  - Point of transverse and lateral restraint at proposed bolt locations
  - 50ksi material
Strength Design

• 9ft beam spacing, Sizing:
  – Bending demand = 31k-ft / ft
  – Required Section = 13.5in³ / ft

• Look at what works…

Strength Design

• Shape Options
  – W-Shape
  – Deck plate is supported, greater shape spacing
  – Inefficient top flange

Strength Design

• Shape Options
  – T-Shape
  – Deck plate acts as top flange (efficient)
  – Shorter deck plate spans between shapes

Strength Design

• T-S
  – Similar to W
  – Additional Torsional Stability
  – Slightly more difficult weld

Strength Design

• Shape Options
  – Channel
  – Inefficient top flange
  – Asymmetric
  – Useful at panel ends to minimize overhang

Strength Design

• Other Shape Options
  – Combination of shapes
  – Continuous Tubes
Strength Design

- Options:
  - Select the most efficient shape:
  - Select the shape for easiest fabrication
- For Proposed Standard:
  - 1/2in to 5/8in deck plate
  - Combine T as the main member with Channel to limit overhangs at deck edges
  - 8in to 12in shape spacing

Deck plate bending
- Longitudinal (primary)
- Lateral (secondary)

Use AREMA allowable stresses

Preliminary Panel

- Proposed:
  - 5/8" Deck
  - Combined T and Channel Shapes
  - 10" shape spacing
  - 16ft panel

Refined Analysis

- Verify Initial design
  - AREMA requirements
  - Additional requirements

Refined Analysis

- Simple deck plate bending
- Buckling of deck plate
- Buckling of supporting shape’s vertical member (e.g. web plate)
- Shear of deck panel

Refined Analysis

- Complex system
- Finite Elements
Refined Analysis

- Evaluate
  - Multi-axial stress
  - Local Stresses
  - Local Buckling
- Fatigue

Non-AREMA evaluations

- Bending of deck + ballast pressure = compression of deck as top flange in bending and with lateral pressure
- Local buckling of deck with ballast pressure and continuous edge restraint
- Fatigue of deck plate to shape weld (global transvers + local lateral bending)

Non-AREMA solution guidance

- Theory of Elastic Stability (Timoshenko, Gere)
- Manual for Design, Construction, and Maintenance of Orthotropic Steel Deck Bridges (FHWA)

Fatigue considerations

- 3 possible controlling conditions
  1. Primary bending of deck section
     a. Shape flange in tension (Cat A)
     b. Shape-to-deck plate weld (Cat B)
  2. Longitudinal bending of deck plate (Orthotropic detail)
  3. Negative bending over supporting structure

Fatigue considerations

- No welds in primary tension region

Showing transvers bending stresses:

Considerations

- Ease of fabrication
- Durable
- Aim for low costs
- Repeatable
- Simplify Connection
### Additional Details

- A word on tolerances
  - Tolerance is associated with Fabrication cost
  - AREMA (Ref.) tolerances for flatness
  - Warping

- Clearly indicate fit-up requirements and expectations
- Shape dimensions and clearances

### Additional Details

- Weight
  - 9in deep timber deck: 780plf
  - 8in Concrete deck: 1400plf
  - Proposed steel deck: 720plf

- Steel deck: Can use existing equipment to install

### Remarks

- However:
  - Get steel deck durability on common spans w/o support brackets and field welding
  - Modular deck assembly
  - Greatly decrease operational costs with repeated deck replacements

- Material Costs
  - Timber at $1750 per mBF: $171/ft
  - Concrete at $1/lb reinforcing & $1500/cy: $1140/ft
  - Proposed deck at $2.00/lb: $1450/ft
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