Wang and Kim *J Infrastruct Preserv Resil* (2022) 3:13
https://doi.org/10.1186/s43065-022-00058-1

Journal of Infrastructure Preservation and Resilience

REVIEW

Open Access

A state-of-the-art review of prestressed concrete tub girders for bridge structures

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Abstract

This paper presents a state-of-the-art review of prestressed concrete tub girders for bridge structures, which are a resilient solution to reduce the occurrence probability of disruptive events and to extend the longevity of built-environments. Technical parameters are categorized in accordance with physical characteristics and corresponding contents are examined. Unlike the case of box girders, the open-section tub girders demand secondary structural elements to improve stability. As far as application is concerned, pre- and post-tensioning methods are taken into account with a focus on the implications of geometric variables, on-site splice, and applied forces. The applicability of existing design provisions is also evaluated in the context of live load distributions and end zone cracking. While the strand size of 12.7 mm and 15.2 mm is dominant in the United States, research has commenced on the use of 17.8 mm strands to accommodate a high level of prestressing force. The effectiveness of precast decking panels is elaborated against conventional cast-in-place decks, and the extension of overhangs is delineated.

Keywords: Bridges, Development, Prestressed concrete, Review, Structural efficiency, Tub girder

Introduction

Prestressed concrete girders are one of the primary structural components in highway bridges and considered to be a resilient solution. Benefits of using precast members include accelerated construction time, favorable life-cycle expense, affordable maintenance and repair, quality control, aesthetics, and minimal disruption to traffic [1, 2]. From a functionality perspective, such bridge configurations enable a long span with durable performance and improved serviceability relative to conventional reinforced concrete members. Prestressed girders were initially constructed in an I-shape; however, the requirements for many girders and aesthetically unpleasant layouts brought about the development of a new girder type [3]. Furthermore, state Departments of Transportation (DOTs) preferred to reduce the number of girders for economic reasons and, as a result, tub girders (also known as U-shaped girders) became a strong alternative, as schematically explained in Fig. 1: because one tub girder can replace two I-beams, cost savings are expected (e.g., fabrication, formwork, in-situ labor, and transport). Local agencies and precasters design, fabricate, and produce various sizes of tub girders. For instance, from 1995 to 2000, the Colorado Department of Transportation (CDOT) spent resources to develop the B618-U series for pretensioning and post-tensioning applications with a web thickness of 125 mm to 254 mm. The B618-U girders, varying from 1,220 mm to 2,440 mm in depth, offer several advantages against closed-box girders in terms of manufacturing process, flexible use of top flanges, and shipping weight.

Over the last three decades, a wide variety of research projects have been carried out to examine the behavior of prestressed concrete tub girders. Some notable subjects are the development of new sections [3], splice details [4], field applications [5], prestressing techniques [6], seismic resiliency [2], strand size [7], ultra-high performance concrete [8], and durability [9]. Notwithstanding these remarkable achievements, a synthesis of contemporary knowledge and practical applications has rarely

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been reported thus far. To link the state of the art of tub-girder research with in-situ implementation, a comprehensive literature review is conducted with an emphasis on geometric configurations, strand details, assessments on existing girders, prestressing methods, splice, decking, overhangs, live load distributions, and end zone behavior. So informed, those who are involved in bridge engineering will better understand recent advances in the design and construction of prestressed concrete tub girders.

Topological configuration

Geometric characteristics

It is recognized that geometric details control the performance of constructed girders. To provide adequate flexural and shear capacities, several factors need to be considered (e.g., girder height, web thickness, and number of prestressing strands). A tub girder is generally composed of a bottom flange and inclined webs plus short top flanges (Fig. 2). Tub girders need drainage holes to allow for the flow of internal water, and cast-in-place concrete for a closure should have a compressive strength of 35 MPa or higher [2]. Tub girders save forming costs, compared with closed box girders, and segmental construction can be implemented for both pretensioned and post-tensioned superstructures at variable web thickness (125 mm to 250 mm). Typically, the web thickness of a tub girder is determined by shear and prestressing forces [6]. It is also expected that a reduction in web thickness increases the girder depth to compensate for its flexural rigidity. Multiple diaphragms may be placed at both ends and the middle of a tub girder [10], as shown in Fig. 3(a), to resist local torsion and relieve stress concentrations. One large or two small neoprene pads can be located underneath a tub girder at the location of piers. The use of end blocks lowers tensile stress in the transverse direction [11], which is beneficial in reducing the likelihood of premature cracking; however, the blocks generate curing heat and delayed ettringite formation (Dunkman 2009). For skewed tub girders, a couple of end block options are available (Fig. 3(b)). The prestressed concrete community

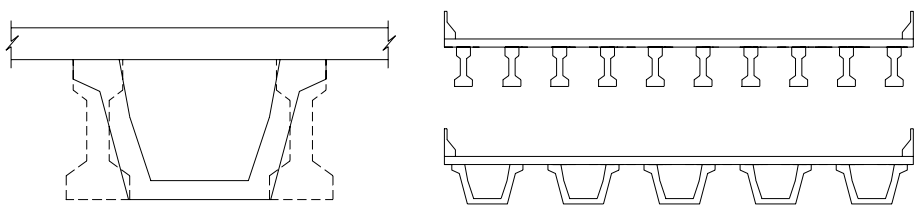

Fig. 1 Conceptual placement of prestressed concrete girders (reproduced from Ralls et al. [3])

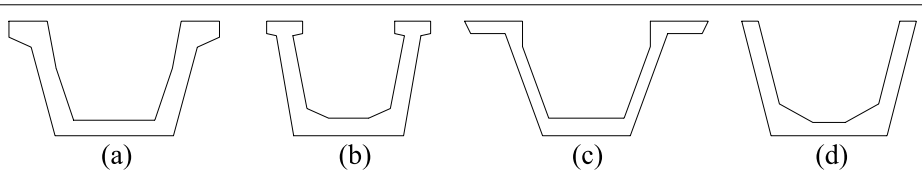

Fig. 2 Selected tub girder sections: **a** Texas U54; **b** Washington UF72G5; **c** Colorado B-618-U; **d** Washington trapezoidal

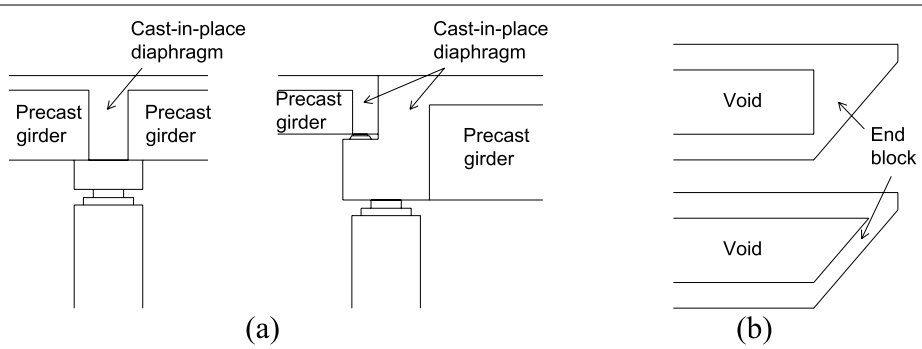

Fig. 3 End details of tub girders (reproduced from Cruz and Wisniewski [5]; TxDOT [13]): **a** cast-in-place diaphragms; **b** end block options

adopts the following structural efficiency factor (Eq. 1) and efficiency ratio (Eq. 2) to evaluate the geometry of a girder section [12]:

$$p = \frac{r^2}{y_t y_b} \tag{1}$$

$$a = \frac{3.46 S_b}{Ah} \tag{2}$$

where p and a are the structural efficiency factor and ratio, respectively; r is the radius of gyration of the girder; y_t and y_b are the distances from the centroid of the girder section to the top and bottom fibers, respectively; S_b is the section modulus for the bottom fiber; and A and h are the cross-sectional area and depth of the girder, respectively. In practice, a bridge system with twin tub girders is often constructed at a deck width varying from 10 to 15 m with a girder spacing between 4 and 8 m on center [6]. The geometry of tub girders is relevant for handling and erection, whereas, when curved girders are transported and erected, stability may be of concern due to a lack of torsional resistance. Installing steel bracings or diaphragms can help increase the torsional stiffness of the open section [14]; particularly, end blocks enhance the degree of force distributions [15].

Girder type

As exemplified in Fig. 2, assorted tub girder sections are used in the United States. These girders possess a typical depth of 1,020 mm to 2,440 mm with a variety of upper flange sizes. Both pretensioning and post-tensioning applications are available. Table 1 enumerates the sectional properties of selected tub girders. Even though the cross-sectional area and moment of inertia of the girders noticeably change with depth, their radii of gyration are relatively stable ($16.32 \le r \le 32.65$). This fact implies that optimal properties exist in tub girders and structurally efficient sections can be proposed in compliance with local bridge specifications. Given that precast girders have standard sections in contrast to cast-in-place girders, design aids are charted to estimate achievable spans with girder spacing [16]. Information on expected costs can be developed as well. It should, however, be noted that these aids are intended for the convenience of preliminary design; hence, practitioners need to check all details associated with actual practices.

Details of reinforcement and strands

Similar to other prestressed concrete girders, 12.7 mm and 15.2 mm strands are broadly used for tub girders at typical spacings of 50 mm. Prestressing forces are released by either flame-cutting or gradual jack-down. The transferred forces bring about elastic shortening and end zone cracking in pretensioned tub girders. Compared with flame-cutting, the gradual jack-down method, performed by releasing a hydraulic pressure, mitigates the occurrence of concrete cracking [17]. In addition to the conventional placement of steel strands in the tensile side of a tub girder that requires a transfer length of 60 times the diameter [18], four to six strands may be positioned in the compression zone of the girder when required [3]. Debonding of prestressing steel reduces stress in the transverse bars [19]. Cast-in-place diaphragms should be adequately reinforced to avoid premature cracking, which is beneficial in maintaining the integrity of spliced girders, and the location of the diaphragms needs to be at least 900 mm away from concrete closures [2].

Steel strands with a diameter of 17.8 mm are employed in tunnel construction, whereas these are not yet widely adopted for highway bridges. Field demonstration projects were reported using 17.8 mm strands alongside preliminary assessments [20]. Because the size of steel strands affects prestressing details, existing design provisions cannot be applicable to members with 17.8 mm strands and currently limited information is available. Expected benefits of using such large diameter strands include reduced number of strands, affordable cost, enhanced construction convenience, efficient fabrication, and increased span length [7, 20, 21]. ASTM A416 [22] specifies that 17.8 mm strands have a nominal area of $A_p = 190$ mm^2 and a weight of $w_p = 14.6$ N/m, which are greater than those of 12.7 mm and 15.2 mm strands ($A_p = 99$ mm^2 and 140 mm^2 with $w_p = 7.6$ N/m and 10.8 N/m, respectively). If the conventional spacing of 50 mm is maintained in a pretensioned girder with 17.8 mm strands, resultant stresses may be overlapped and premature cracking can take place [21].

If the diameter of strands is changed from 15.2 mm to 17.8 mm, several aspects of tub girders need to be examined (e.g., achievable span length, girder depth and spacing, and bond performance). The configuration of tub girders dominates the effectiveness of 17.8 mm strands [23]; as such, by adopting 17.8 mm strands in a tub girder, its overall depth would be reduced, thereby lowering the self-weight of the superstructure with an increased span length. It should, however, be noted that the use of 17.8 mm strands may not significantly extend the achievable span length of tub girders [7]. The number of 17.8 mm strands can be increased in a tub girder until its stress exceeds the serviceability limits stipulated in bridge specifications. If necessary, partial debonding may be considered or the strands may be harped. The provisions of conventional codes, such as ACI 318 [24] and AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications (BDS) [18],

Table 1 Properties of tub girders

State/agency	ID	Depth (mm)	Area (x10^{3}mm^2)	Inertia (m^4)	y_t (mm)	y_b (mm)	S_t (x10^{6}mm^3)	S_b (mm^3)	R (mm)
CA	UB 1400	1,397	864	21	787	610	243	313	18.54
	UB 1550	1,549	926	28	864	685	290	368	20.52
	UB 1700	1,702	988	36	940	762	340	426	22.47
	UB 1850	1,854	1,050	45	1,016	838	394	488	24.40
	UB 2000	2,007	1,112	55	1,092	915	451	552	26.31
	UB 2150	2,159	1,174	67	1,168	991	511	620	28.20
CO	U48	1,219	903	19	610	609	278	281	17.11
	U60	1,524	1,022	34	762	762	394	399	21.41
	U72	1,829	1,142	53	914	915	524	530	25.58
	U84	2,134	1,261	79	1,067	1,067	666	674	29.63
FL	FU48	1,219	778	15	737	482	185	270	16.32
	FU54	1,372	823	20	813	559	225	327	18.56
	FU63	1,600	888	30	940	660	291	417	21.88
	FU72	1,829	954	43	1,067	762	364	512	25.13
PCI	PCI72_9	1,829	1,311	57	1,016	813	510	634	24.73
	PCI84_9	2,134	1,458	86	1,168	966	661	807	28.71
	PCI96_9	2,438	1,601	122	1,321	1,117	827	994	32.65
	PCI72_10	1,829	1,410	61	1,016	813	544	662	24.51
	PCI84_10	2,134	1,569	91	1,168	966	707	845	28.48
	PCI96_10	2,438	1,729	130	1,321	1,117	886	1,044	32.36
TX	U 40	1,016	632	8	610	406	127	184	13.67
	U 54	1,372	723	19	813	559	209	295	18.97
	U72_9	1,829	1,314	57	1,016	813	510	634	24.70
	U84_9	2,134	1,458	86	1,168	966	661	807	28.71
	U96_9	2,438	1,601	122	1,321	1,117	827	994	32.65
	U72_10	1,829	1,410	61	1,016	813	544	662	24.51
	U84_10	2,134	1,569	91	1,168	966	707	845	28.48
	U96_10	2,438	1,729	130	1,321	1,117	886	1,044	32.36
WA	U54_4	1,372	670	14	838	534	145	229	16.78
	U66_4	1,676	780	24	1,016	660	214	320	20.68
	U78_4	1,981	889	38	1,168	813	295	423	24.50
	U54_5	1,372	717	15	889	483	149	264	16.82
	U66_5	1,676	826	26	1,041	635	222	361	20.80
	U78_5	1,981	935	41	1,194	787	306	474	24.71

CA California, *CO* Colorado, *FL* Florida, *PCI* Prestressed Concrete Institute, *TX* Texas, *WA* Washington

y_t and y_b = distances from neutral axis of girder to top and bottom, respectively; S_t and S_b = section moduli for top and bottom components, respectively; r = radius of gyration

cannot be used for the prediction of transfer length in a beam with 17.8 mm strands [21]. Further research is recommended in regard to the application of 17.8 mm strands (e.g., bond behavior, transfer length, spacing requirements, partial debonding, harping, and splitting cracks). Aligning with these research efforts, precasters need to examine their jacking apparatus and harping devices in order to hold down the relatively stiff strands.

Overhangs, shoulders, and drainage

Bridge overhangs extend the functional width of a deck beyond exterior girders and resist dead (deck, barrier, and railing) and live (vehicle, pedestrian, and barrier impact force) loads. The length of an overhang should be between 1.2 m and 1.8 m [10, 25]. Contingent upon girder spacing, transverse and longitudinal reinforcing bars are placed at a maximum spacing of 114 mm to 305 mm in overhangs [25]. A minimum gap between the ends of the top flange and overhang should be 150 mm in order to avoid water dripping [25]. The overhangs of a composite tub girder influence distortional warping stress and, if a single traffic lane is loaded, the warping stress noticeably increases [26]. Torsional moments created by overhang loadings need to be considered in a superstructure

design. Excessive overhangs should be avoided to control the torsional and bending stresses of the cantilever region. In a curved tub girder system, overhangs significantly raise the torsional stiffness of the superstructure [27]. Barriers in overhangs stiffen the superstructure behavior owing to the increased moment of inertia; however, this effect is generally ignored for design convenience. While a single cell girder with wide overhangs is beneficial for ramp structures, the overhangs' intolerable deflections and rotations are of concern.

The range of shoulders is between 300 mm and 3,000 mm for superstructures carrying one- to four-lanes of traffic [28]. Considering the safety and mobility of traffic, the width of shoulders should be sufficiently large and previous accident records are a good source for the determination of a shoulder size. Frequent intervals of deck drains are necessary if full shoulders are not provided [29]. Some Departments of Transportation (DOTs) regulate the inclusion of shoulders based on the volume of design traffic (e.g., Michigan DOT requires that the minimum clear width of a bridge be a traveled way plus shoulders when average daily traffic is over 2,000 [30]).

A bridge drainage system, consisting of grates, inlets, pipes, and gutters, is an important component to properly manage runoff and remove waters from bridge decks. In many cases, the Hydraulic Engineering Circular No. 21 (HEC-21) of the Federal Highway Administration [31] is referenced when designing the drainage of highway bridges. Deck drainage systems should be away from expansion joints and bearings [25] in order to enhance the safety and longevity of bridges.

Placement and in-situ fabrication

Pretensioning and post-tensioning methods

Prestressing methods for precast tub girders involve both pre- and post-tensioning with straight and harped strands. Regarding implementation cost, pretensioning has economic advantage over post-tensioning [32]. Pretensioning is performed in a stressing bed comprising abutments, a jacking apparatus, hold-downs, and steel strands. Steam curing accelerates the hydration of concrete and casting operations. Post-tensioning is conducted to achieve a continuity between adjacent tub girders [5] before and after casting deck concrete using special equipment. Dependent upon the number of spans and intermediate supports, multiple tub girders may be post-tensioned and spliced to establish a full continuity [33]. The benefits of post-tensioned splices are found in the reduced number of piers and joints, favorable maintenance expense, enhanced serviceability, shallowd superstructure geometries, and increased girder spacing [33]. The following parameters are recommended to be checked at the design stage: concrete properties, prestressing forces, and long-term deformations. To determine the post-tensioning option, several factors need to be considered [34]: span length, girder spacing, self-weight, maintenance, lateral stability, and life cycle costs. When a girder section is not large enough to accommodate prestressing forces, partial-length post-tensioning is conducted rather than full-length tensioning [35]. Pursuant to customary design equations, short- and long-term prestress losses are calculated, covering anchor set, friction, elastic shortening, relaxation, creep, and shrinkage [18].

Splice

Like other superstructure members, multiple pieces of tub girders are erected and spliced [36]. The diaphragms of tub girders are often positioned at splice locations. Even if the length of precast girders is typically limited to 45 m on account of handling challenges (a girder weight of 85 to 120 tons is acceptable for shipping [37]), the design and construction of long-span bridges at affordable cost have been of interest. By connecting multiple prestressed concrete girders on site, such an intended plan is accomplished and the serviceability of the superstructure is enhanced; scilicet, after splicing the girders, the superstructure system becomes more effective in carrying live loads with reduced deflections and stresses. In addition, a number of advantages are expected from spliced girders in comparison with segmental girders: moment redistribution, reduced maintenance, favorable labor, and cost savings [38]. From a functionality standpoint, the location and methods of splicing are considered important. Connection details should thus be examined to select suitable splicing techniques, incorporating efficient strand profiles, hardware usage, tensioning method, and contractor's specialty [35]. Various splice types were proposed and implemented for prestressed concrete girders (Fig. 4). Girder-splicing may be conducted on piers or in span, depending upon structural need. The former is common and economical, whereas the latter is effective in increasing span length. To facilitate splicing in the field, prestressing strands and steel reinforcing bars are extended from the ends of pretensioned girders. For the case of post-tensioning, strands are passed through hardened splice diaphragms and tensioned; then, the ducts are grouted.

Temporary supports are necessary when erecting and splicing prestressed concrete girders. As reported in a case study on the Interstate 15 Bridge over 4500 South Street in Salt Lake City, Utah [42], prestress losses occurred gradually with time and relatively large magnitudes were recorded before grouting ducts and after placing diaphragms. These observations indicate that care should be exercised to preserve prestressing forces when

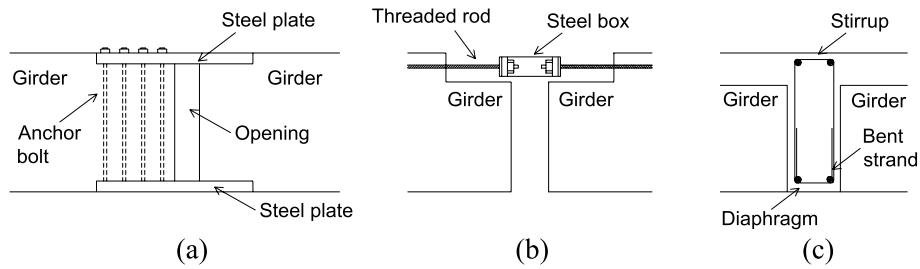

Fig. 4 Splice methods for continuity of pretensioned girders (reproduced from Bishop [39]; Sun [40]; Miller et al. [41]): **a** bolted plates; **b** threaded rods; **c** positive moment connection

in-situ splicing is conducted. The failure of spliced tub girders was often visible at a pier location, where extensive cracking took place with inelastic deformations [4]. Damage localization was conspicuous in the splice diaphragm and the girders did not exhibit apparent cracks, corroborating the critical region of the spliced tub girder system. Splice connections may be prestressed outside support locations to achieve a long-span superstructure [38]. Fully prestressed splices are tensioned with either strands or bars, and the space between the adjacent girders is grouted. Partially prestressed splices contain hooked bars, also known as 180-U bars, in addition to prestressing strands.

Full- and partial-depth decks

Deck slabs are an important element in a bridge, transferring live loads to the supporting girders. Decks can be cast either on site or in a plant (cast-in-place and precast, respectively). Cast-in-place decks are prevalent; however, its construction is slow, which can be addressed by the use of precast decks. Cost-savings are also achieved in the case of precast decks. Precast concrete panels may be used to cover tub girders, so that the load-bearing system acts like a closed-box superstructure showing composite action [43]. It should be noted that stay-in-place metallic decking is utilized to accommodate deck slabs between tub girders [10]. The application of precast decks is categorized into full- and partial-depth panels. Contrary to the full-depth panels enveloping the entire deck area, partial-depth panels function as a stay-in-place element to be covered by cast-in-place concrete. Caution should be taken at the location of connections between the partial-depth panels, where excessive deflections are associated [44]. Post-tensioning is available for full-depth precast decks in the transverse and longitudinal directions [45].

Behavioral aspects

Live load distribution

The partial occupancy of vehicle loadings causes uneven live load distributions. Girders adjacent to live loads are subjected to a higher stress than those away from the loading. When a superstructure is loaded with more trucks, discrepancies between the supporting girders tend to diminish [46]. The geometric configuration of slab-on-girder bridges is salient for the distribution of a live load, which is related to mechanical interactions among the deck, girders, and other load-carrying elements [47]. Simple distribution factors were conventionally used for the design of a superstructure (e.g., S/D format, where S is the girder spacing and D is an empirical constant [48]). Since the first edition was published in 1994, AASHTO LRFD BDS [18] has prescribed case-specific equations for live load distributions. The equations were calibrated with several variables (e.g., superstructure type, girder spacing, slab thickness, span length, number of loaded lanes, and stiffness) and empirical expressions were developed [49]. The distribution equations are multiplied by bending moments and shear forces determined from beam-line analysis in order to attain the magnitudes of the live load on individual girders.

Truck loadings are positioned to generate the maximum moment and shear in tub girders (Fig. 5(a)). Usually, the placement of vehicle loadings near exterior girders is more sensitive than interior girders [50]. As graphed in Figs. 5(b) and (c), the distribution factors for bending moments in the exterior girders noticeably changed with the number of loaded lanes and the factors for shear remained almost constant; on the interior girders, the shear distribution factors were higher than their moment counterparts. These observations justify the load categories specified in AASHTO LRFD BDS (i.e., one-lane- and multiple-lane-loaded cases). The number of diaphragms and lateral bracings is not influential in distributing live load [51], whereas these elements are still necessary for the stability of a superstructure system [46]. As long as reasonable stiffness is given to deck slabs, their thickness does not alter live load distribution factors [52]. The continuity of girders marginally affects live load distributions. Nutt et al. [53] reported a difference of about 10% between simply-supported and continuous

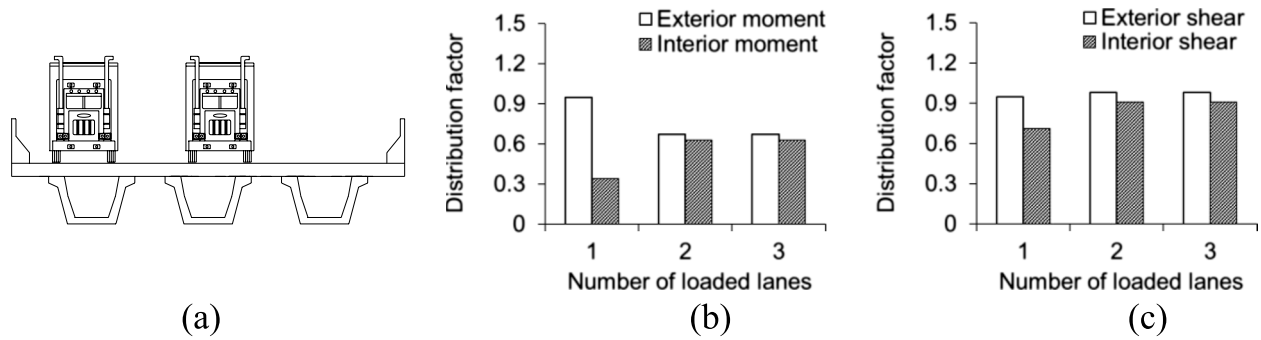

Fig. 5 Live load distribution factors in tub girder bridge (reproduced from Mott and Diaz [55]): **a** positioning of truck loads; **b** moment factors with a span length between 65 and 120 ft; **c** shear factors with a span length between 65 and 120 ft

Table 2 Distribution factors of a tub girder bridge (reproduced from Hughs and Idriss [57])

Method	AASHTO Standard	AASHTO LRFD BDS	Finite element modeling
Interior girders			
Moment	0.716	0.555	0.565
Shear	0.564	0.890	0.791
Exterior girders			
Moment	0.808	0.966	0.550
Shear	0.808	1.065	0.714

bridges. If a bridge is heavily loaded, distribution factors decrease [54] because of a modified load path.

While live load distribution factors are estimated according to design equations [18], computer models (e.g., finite element analysis) can be employed to determine refined distribution factors. The empirically calibrated design equations may not provide accurate information on live load distribution factors, as claimed by Petty [56]: the AASHTO equations overestimated load effects on interior girders. Hughs and Idriss [57] evaluated the applicability of live load distribution factors calculated per AASHTO LRFD BDS [18]. Finite element modeling was conducted with a benchmark bridge superstructure supported by six prestressed concrete tub girders (1.4 m deep). The applied truck load consisted of three axles: 67 kN (front), 93 kN (middle), and 93 kN (rear). As summarized in Table 2, for interior girders, the distribution factors of AASHTO LRFD BDS [18] agreed better than those of the AASHTO Standard Specifications [48]; however, for exterior girders, both of the AASHTO specifications revealed conservative factors. It was recommended that field data be acquired to recalibrate the equations of tub girders for shear distribution factors.

End zone response and design approach

When a prestressed girder is tensioned and the force is transferred, multiple cracks develop due to stress concentrations in the end region (Figs. 6(a) and (b)). Splitting cracks occur in pretensioned members that are reliant upon the bond between the strands and concrete, while bursting cracks related to stress distributions degrade the performance of both pre- and post-tensioned members. The formation of these bursting and spalling cracks is either instantaneous, right after prestress transfer, or time-dependent within a few weeks from the transfer [58]. The fact that many existing tub girders in the United States were empirically developed without physical testing leads to premature cracking in the end regions [15]. Figure 6(c) depicts typical shear stress distributions in the end region of a tub girder [10]. The high stresses at the strand level (e.g., cell numbers of 2, 6, 10, 14, and 18) cause bursting cracks, especially within the transfer length. The spalling stress of a girder dwindles with the increased distance from the end [59]. In accordance with the Prestressed Concrete Institute (PCI) Committee on Quality Control Performance Criteria [60], several factors induce end region cracking: improper design and fabrication, concrete settlement, differential stresses between the web and flange, and insufficient cover depth. Specifically, at the girder level, the following parameters are responsible for end zone cracking: geometric details, strand arrangement and amount, prestressing force, concrete strength, thermal and shrinkage stresses, and curing [17]. Environmental factors can accelerate the cracking of end zones (e.g., temperature and drying shrinkage [15]).

When end region details are inadequate, bursting cracks are observed with complex stress states [10]. The width of these cracks is around 0.13 mm to 0.30 mm [61, 62] and, if wider than 0.18 mm, epoxy injections may be necessary [17]. In a pretensioned girder, the maximum moment of the end zone takes place near the centroid of the section

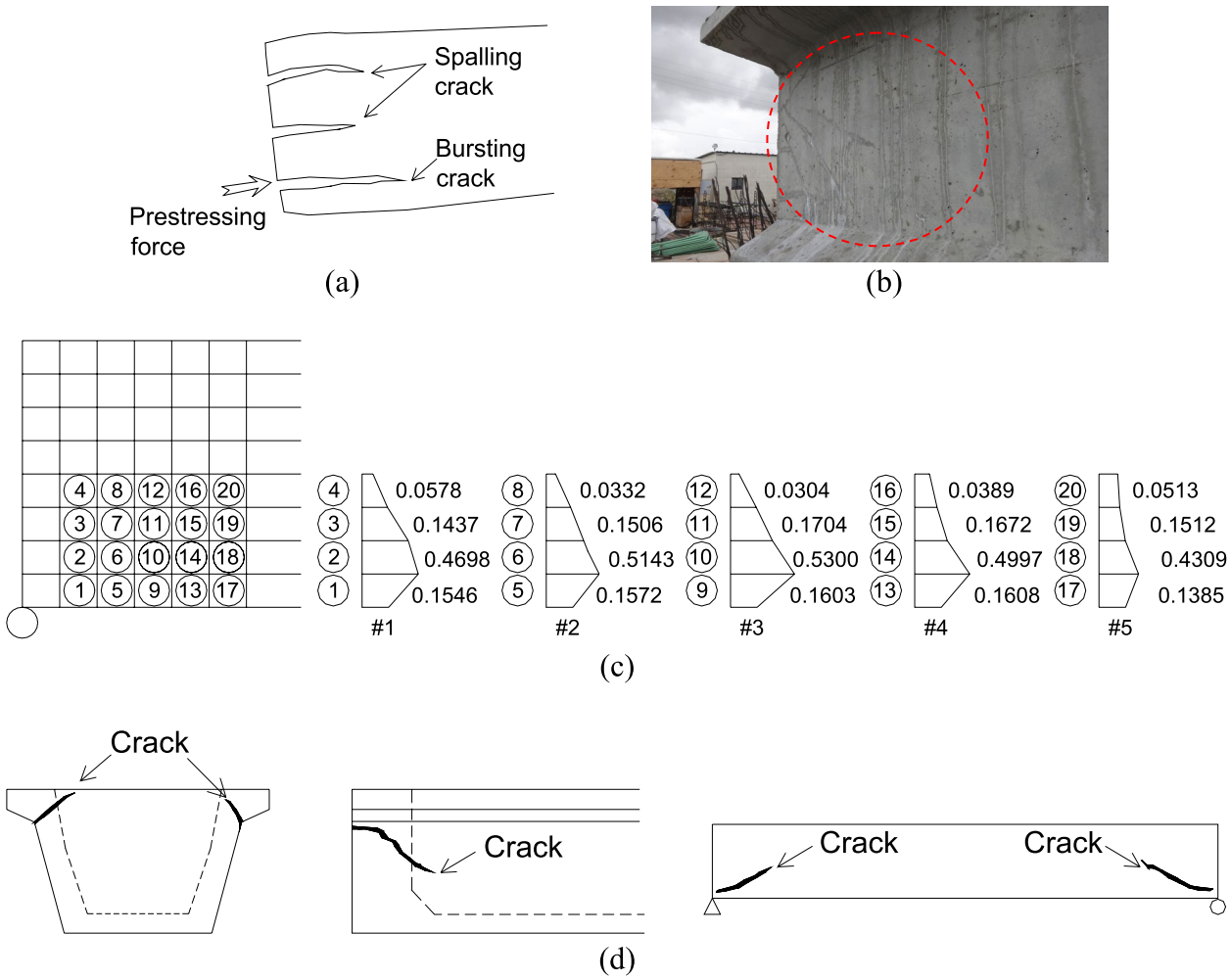

Fig. 6 Bursting and spalling at end-region of prestressed concrete girder (reproduced from Barrios [58]; Huang and Shahway [10]; photo courtesy Y.J. Kim): **a** crack type; **b** occurrence; **c** Stress distribution in end-region; **d** short-term and web cracking during construction

where spalling cracks form [15], which redistribute the prestressing forces to nearby concrete [63]. Web-cracking may develop during the construction of tub girders, albeit uncommon, and sometimes cracks initiate at the junctions between the web and flanges owing to stress concentrations, rather than prestressing forces (Fig. 6(d)). For the purpose of crack control, in addition to shear stirrups near the end of girders, transverse reinforcement is provided: vertical and lateral steel bars are necessary to alleviate the occurrence of spalling and bursting cracks. Although the behavior of an end zone becomes unstable due to cracking, the stress level of the transverse reinforcement is generally lower than its yield stress [15].

Typical parameters affecting stress development in the transverse reinforcement are strand patterns, prestressing levels, concrete thickness, and bar size [59]. For the design of end region reinforcement, a couple of approaches are used, namely, finite element analysis [10, 64], analytical expressions [65], strut-and-tie modeling [66], and empirical equations [18]. Each approach has positive and negative facets. Finite element analysis generates detailed data in regard to stress and concrete cracking, whereas it requires significant endeavors for constructing a girder model. Strut-and-tie modeling is semi-theoretical and determines reinforcement with simplified force-transfer trajectories. The use of analytical and empirical equations is convenient; however, the level of accuracy is ambiguous and may cause a problem after erecting girders. To address bursting and spalling cracks in the end zone of a tub girder, alternative reinforcing schemes can be considered [19]. With the increased diameter of steel strands from 12.7 mm to 15.2 mm or 17.8 mm, end zone cracking became more prominent [17]. As such, conventional approaches for designing the end zone of a tub girder should be revised if 15.2 mm and 17.8 mm strands are used.

Under the allowable prestressing force stipulated in local bridge design specifications, the amount of transverse steel bars is determined to inhibit bursting and spalling cracks, including a due consideration on constructability. Experience-based design is commonplace: AASHTO LRFD BDS [18] requires that the stress level of reinforcing bars be less than 138 MPa, within $h/4$ of the girder end (h = girder depth), when subjected to at least 4% of the prestressing force. The $h/4$ requirement is intended to preclude spalling cracks near the centroid of a girder section, and the 4% resistance is based on the assumption that a ratio between the girder height and transfer length is 2.0 [59]. The AASHTO stress limit of 138 MPa was frequently exceeded [15, 62, 67], which illustrates that the uniform stress distribution assumed in AASHTO LRFD BDS [18] is inaccurate and should be reassessed with realistic stress profiles. In some cases, the implementation of stress control is not consistent. For example, the stress of transverse reinforcement is limited to 124 MPa in Florida [68] and a design stress of 207 MPa is stated in the PCI design manual [69].

Summary and conclusions

This paper has discussed a holistic overview of prestressed concrete tub girders for highway bridges. The geometric configuration and reinforcing schemes of the girders dominated the performance of a superstructure. Both pretensioning and post-tensioning were applicable in practice. Several decking methods were proposed and implemented. The distribution of live load was studied in line with an assessment of the existing AASHTO equations. The majority of research concerning end zone cracking was empirical and inconsistent contents were documented in the literature. The following conclusions and recommendations are drawn:

- The behavior of tub girders is controlled by several parameters (e.g., geometric properties, prestressing schemes, and in-situ splice). Diaphragms are required to enhance the stability of the open-section girders against torsional stress. A couple of end block options are available in skewed tub girders to address curing heat and delayed ettringite formation. The geometric details of tub girders vary from agency to agency; accordingly, an appraisal is needed to investigate the structural efficiency of existing girders. In so doing, parameters influencing the efficiency of tub girders are identified and will be considered when developing a new girder series.
- Across the United States, the strand size of 12.7 mm and 15.2 mm is utilized for prestressing girders. Harping and debonding techniques are performed to reduce stress levels near the end of the girders. Early efforts encompassed the application of 17.8 mm strands, whereas generalized design guidance was not yet developed. For the adoption of the 17.8 mm strands, research is indispensable to check the applicability of current design specifications and to propose new provisions. Precasters' facilities should be examined as well.
- Tub girders are post-tensioned and spliced on site to accomplish continuities, which will benefit the superstructure system. Before planning a post-tensioning scheme, the designer should evaluate span length, girder spacing, self-weight, maintenance, stability, and life cycle costs. The loss of prestress may be estimated according to AASHTO LRFD BDS. Splicing redistributes bending moments and improves serviceability.
- Full- and partial-depth decks are used for tub girders. Cast-in-place concrete is conventional, while precast panels offer a number of advantages as regards construction time, quality, and costs. After placing decks, a composite system is achieved and thus the tub girders act like closed boxes. Overhangs extend the space of the deck, while their length should be determined within a range that does not cause excessive deflections and distortional stresses.
- The interaction among the deck, girders, and secondary structural members of tub girders results in uneven live load distributions. The position and number of vehicles govern the magnitude of the distribution factors. The empirically calibrated distribution factors of AASHTO LRFD BDS are convenient for preliminary design, whereas refined factors attained from finite element modeling provide accurate live load effects. The shear distribution factors of AASHTO LRFD BDS are recommended to be recalibrated for tub girders.
- End zone cracking is a problem that remains unsolved. The splitting and bursting cracks of prestressed concrete girders frequently occur. Design approaches were based on experience and their applicability is largely unknown; consequently, discrepancies exist in the bridge engineering community. Extensive endeavors should be expended to ameliorate practice guidelines.

Acknowledgements

The authors gratefully acknowledge support from the Colorado Department of Transportation (CDOT). The contents of this paper reflect the views of the authors and do not necessarily reflect the official views of CDOT or any other agencies.

Authors' contributions

JW and YJK conceived the scope and contents of the research and wrote the manuscript. The author(s) read and approved the final manuscript.

Funding

Not applicable.

Availability of data and materials
The datasets used and/or analyzed during the current study are available from the corresponding author on reasonable request.

Declarations

Ethics approval and consent to participate
Not applicable.

Consent for publication
Not applicable.

Competing interests
The authors declare that they have no competing interests.

Received: 2 September 2022 Revised: 28 September 2022 Accepted: 1 October 2022
Published online: 14 October 2022

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