Jacking Steel Bridge Superstructures in Washington State

JOHN A. VAN LUND

Lifting steel bridge superstructures in Washington State are described. The use of jacks is an effective means for raising existing plate girder bridges to increase vertical clearance, to remove and replace defective bridge bearings, and to transfer the superstructure dead load from one existing bridge substructure to another. Case studies are presented for two successful jacking operations that involved connecting permanent steel jacking beams to the girders and jacking upward from the substructure. Generally, data on jacking operations do not appear in the technical literature. In one case study presented, data were recorded from jack gauges, and a comparison between calculated and recorded lifting loads is presented that indicates the transverse distribution of loads to the jacks. The total lifting loads recorded exceeded the calculated loads by 22 to 32 percent. Possible reasons for these discrepancies and recommendations for lifting bridge superstructures are given. It is recommended that jacks be sized for at least 1½ times the calculated lifting loads.

During new bridge construction, the lifting of heavy steel superstructure and concrete segments has been accomplished by floating the structural elements to the job site on barges and lifting them from above with long cables or rods and center-pull jacks. Some notable examples of heavy superstructure lifts follow.

EXAMPLES OF HEAVY SUPERSTRUCTURE LIFTS

Fremont Bridge, Willamette River, Portland, Oregon

In March 1973, the steel tied-arch center span, 900-ft (274-m) long, weighing 6,000 tons (5,443 tonnes), was lifted into place 160 ft (49 m) above the Willamette River in Portland, Oregon. Thirty-two 200-ton (181-tonne) center-hole jacks and threaded rods of 4-in. (102-mm) diameter were used. The total time for the lifting operation was 40 hr. It was reported to be the longest and heaviest span lifted in the world (*1*).

Ponte Presidente Costa e Silva, Guanabara Bay, Brazil

The Ponte Presidente Costa e Silva or the Rio Niteroi Bridge over Guanabara Bay connects the cities of Rio de Janeiro and Niteroi, Brazil. The navigation spans consist of parallel steel box girders with two side spans of 656 ft (200 m) and a center span of 984 ft (300 m).

The side spans were divided along the bridge centerline into two equal sections that were barged to the site and lifted 172 ft (52.5 m). Each side span section was 958 ft (292 m) long and weighed 2,480 tons (2,250 tonnes). The two sections were lifted concurrently by ring girders, jacking columns, and twelve 500-ton (450-tonne) jacks. The total load lifted for each side span was 5,815 tons (5,275 tonnes) and was repeated for the other side span. The time for lifting the side span sections was 84 hr. Finally, the 577-ft (176-m) center span, weighing 3,770 tons (3,420 tonnes) was lifted 220 ft (67 m) from the water level by eight 500-ton jacks and was accomplished in 6 days. This lifting operation was reported to be the second-heaviest bridge jacking operation (*2*).

Quebec Railway Bridge, St. Lawrence River, Quebec, Canada

In September 1917, the 640-ft (195-m) suspended span of the Quebec Railway Bridge, weighing 5,400 tons (4,900 tonnes), was successfully lifted into place 150 ft (46 m) above the St. Lawrence River. An earlier attempt failed on September 11, 1916, when the span was dropped after it had been lifted 12 ft (3.7 m) and 11 workers perished. The raising of the span, which took 4 days to complete, was the heaviest bridge jacking operation in the world until the center span of the Fremont Bridge was lifted in 1973 (*3,4*).

LIFTING EXISTING BRIDGE SUPERSTRUCTURES

Existing bridge superstructures are lifted for a variety of reasons. Road resurfacing on Interstate highways has reduced the vertical clearances on many of the overhead bridges. Two solutions are possible: cut and lower the roadway below, or raise the bridges. According to Ramey (*5*), raising the bridges to obtain adequate vertical clearance may be more economical.

Bridge superstructures are also raised to rehabilitate and replace bearings on older bridges, or in the case of newer bridges, to replace defective pot bearings (*6*). In Washington State, existing bridge superstructures have been lifted to improve steep approach grades, and to remove and replace bearings. As noted in the Brazilian case study, the dead load of an existing bridge superstructure was lifted from its existing piers and transferred to a new steel-arch supporting system by jacking.

Washington State Department of Transportation, Transportation Building, Olympia, Wash. 98504.

CASE STUDY 1: FRANKLIN FALLS BRIDGE, SR-90, KING COUNTY, WASHINGTON

The Franklin Falls Bridge is located approximately 50 mi (80 km) east of Seattle, Washington, on westbound Interstate 90. This continuous, steel-plate girder bridge has two spans of 350 ft (107 m) each (Figure 1). The four plate girders are 13 ft (4 m) deep and are spaced at 14 ft (4.3 m) centers. Originally built in 1971, the bridge was raised in 1978 during the construction of the Denny Creek Bridge.

The bridge was jacked 20.6 ft (6.3 m) at Pier 1, 17.1 ft (5.2 m) at Pier 2, and 13.6 ft (4.1 m) at Pier 3 to provide additional vertical clearance for avalanches that pass below the bridge. The raising resulted in a 4 percent final grade across the bridge, which was less than the original 5 percent grade.

Jacking

At the end piers or abutments, jacking stiffeners were added 2.25 ft (0.7 m) from the existing girder bearing stiffeners so that the four girders could be jacked concurrently. The total dead load to be lifted at each end pier was 700 tons (636 tonnes). Four 300-ton (272-tonne) jacks, one jack per girder, were used to lift the bridge superstructure at the end piers. The ratio of total jack capacity to total calculated dead load was 1.7.

At the intermediate pier, Pier 2, jacking diaphragms consisting of plate girders 75 in. (1,905 mm) deep were connected with high-strength bolts (ASTM A490) of ⅞-in. (22-mm) diameter to the existing bearing stiffeners in the exterior bays (Figure 2). The total dead load to be jacked was 2,500 tons (2,268 tonnes). Sixteen 300-ton jacks, four jacks per lift point, with a total capacity of 4,800 tons (4,354 tonnes) were used to lift the superstructure. The ratio of total jack capacity to total calculated dead load was 1.9.

Jacking Sequence and Deflection Limitations

The jacking was done in four or more stages as shown in Figures 3a and 3b. When each stage was completed, concrete was placed in 4-ft (1.2-m) lifts and cured before proceeding to the next jacking stage.

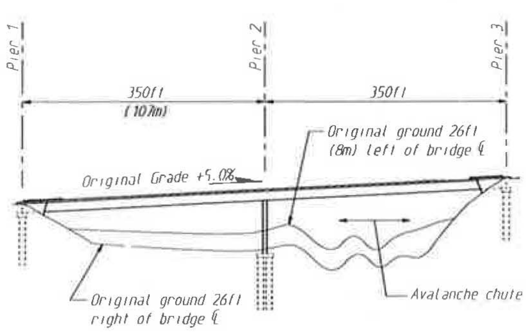

FIGURE 1 Elevation of Franklin Falls Bridge before jacking.

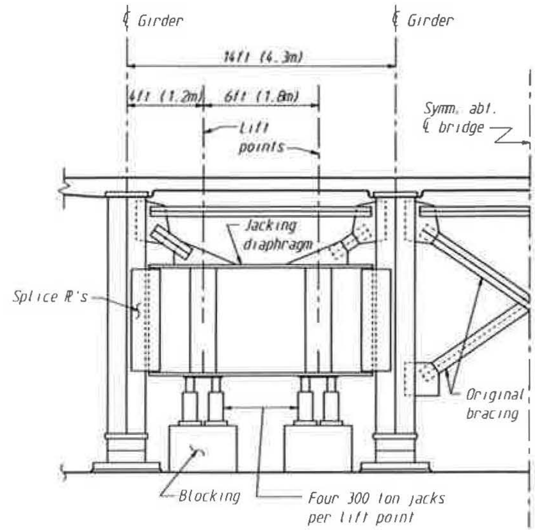

FIGURE 2 Jacking diaphragms at the intermediate pier, Pier 2.

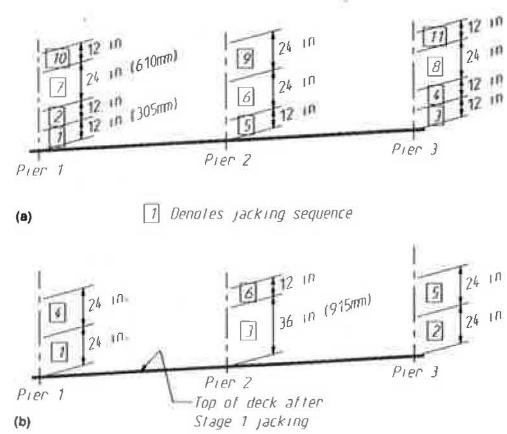

FIGURE 3 Jacking (a) Stage 1, and (b) subsequent stages.

Vertical deflection limits were established to avoid overstressing the concrete deck, steel girders, and crossframes. These were not to be exceeded during any jacking sequence. The relative vertical deflection between any girder at a pier location was $\pm 1/4$ in. (6.4 mm). During end pier jacking, the upward deflection between the end piers and intermediate pier, Pier 2, was not to exceed 24 in. (610 mm). When the superstructure was jacked at Pier 2, the deflection was not to exceed 12 in. (305 mm) relative to the end piers. These vertical deflection limits are shown in Figures 4a and 4b.

Bracing

No jacking was permitted when the wind velocity exceeded 25 mph (40 km/hr). To prevent the bridge from shifting or swaying, transverse restrainer struts were connected between the exterior girders and concrete side walls at each end abutment. At Pier 2, vertical cantilever wind-bracing pipes were

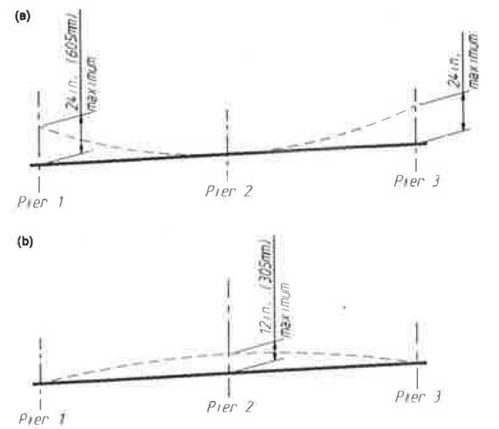

FIGURE 4 Vertical deflection limits for (a) jacking at Piers 1 and 3, and (b) jacking at Pier 2.

attached to the pier top and enclosed by a wind-bracing frame attached to the girders in the center bay.

Eight longitudinal tie rods 2 in. (51 mm) in diameter were connected to the bottom flange of the jacking diaphragms and the concrete deck to ensure that the jacking diaphragms would remain plumb. These were located at each lift point.

CASE STUDY 2: CAPITOL BOULEVARD BRIDGE, SR-5, OLYMPIA, WASHINGTON

The Capitol Boulevard Bridge over Interstate 5 was originally built in 1957 and was reconstructed in 1987–1988 (7). The existing steel-plate girder superstructure was retained, and a new steel-arch supporting system was constructed to replace the existing concrete piers that were removed so that the Interstate below could be widened from four to eight lanes.

One of the most interesting and challenging engineering problems involved the transfer of the existing superstructure dead load to the new arches by jacking.

Bearing Replacement at End Piers 1 and 4

Jacking Sequences 1 and 2

Jacking was done at one location at a time beginning at Pier 1 followed by Pier 4 (Figure 5). Eight 100-ton (91-tonne) hydraulic jacks were used to raise the superstructure at the end piers so that the existing steel slide bearings could be removed and replaced with new ones. As shown in Figure 6, permanent steel jacking beams, W16 × 57 (ASTM A588), were connected to the existing girders with ⅞-in.-diameter high-strength bolts (ASTM A490). Lift points for jacks were located 2.25 ft (0.7 m) from the girders. The hydraulic system used is shown in Figure 7.

The maximum vertical deflection permitted at the end piers was 2.5 in. (64 mm) to prevent overstressing the girders during jacking. In the transverse direction, the relative vertical displacement between adjacent girders was not to exceed ± ⅛ in. (± 3.2 mm), and the relative vertical deflection of the jacks

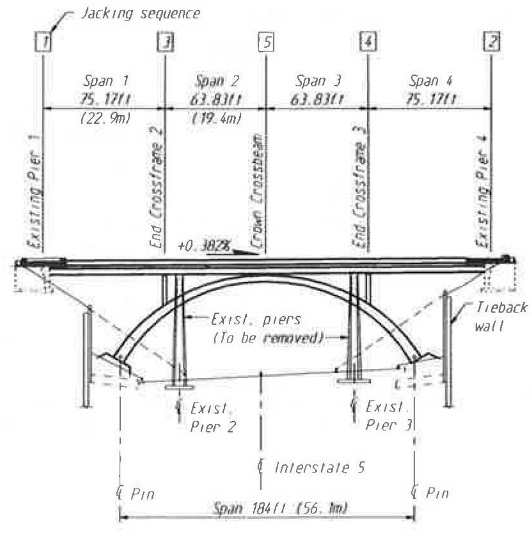

FIGURE 5 Elevation, Capitol Boulevard Bridge.

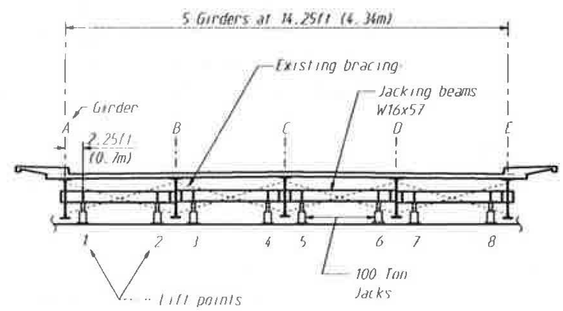

FIGURE 6 Section at end piers.

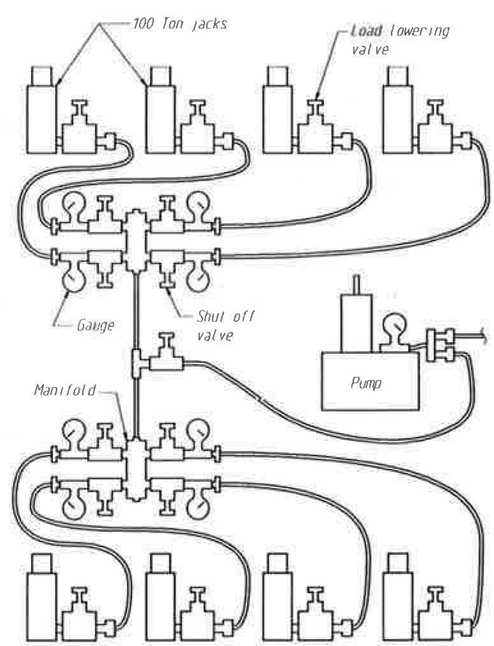

FIGURE 7 Hydraulic jacking system.

on each side of a girder was not to exceed ±⅙ in. (±1.6 mm). These limits on vertical displacement were established so that the original 30-year-old concrete deck, which was to be retained, would not be overstressed.

Calculated Lifting Loads at End Piers

The total lifting load at the end pier was calculated for the original three-span continuous structure shown in Figure 8 by the computer program STRUDL. The noncomposite stiffness of the five steel girders was used because there were no shear connectors and the concrete deck had a joint 1-in. (25-mm) wide over each pier.

The end pier reaction was 300 kips (1,334 kN) for the span dead loads, 20 kips (89 kN) for the concentrated dead load at the centerline of bearings, and 20 kips to lift the system 2.5 in. (63.5 mm). The total calculated lifting load was 340 kips (1,512 kN).

The transverse distribution of the total calculated lifting load to the jacks was determined by applying five equal loads of 68 kips (302 kN) at each girder.

Results of End Pier Jacking

The calculated and recorded lifting loads at each jack are presented in Table 1 and are also expressed as a percent of the total. The total loads recorded were 417.6 kips (1,857 kN) at Pier 1 and 447.8 kips (1,992 kN) at Pier 4, which were 23 and 32 percent greater than those calculated. When expressed as a percentage, the transverse distribution of the total lifting load exhibited good correlation with calculated values for the exterior jacks. However, the actual transverse distribution of the lifting load to the interior jacks was not consistent between Piers 1 and 4 and did not correlate well with the calculated values.

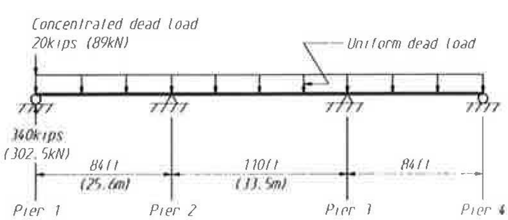

FIGURE 8 Original spans, end pier jacking.

Load Transfer at End Crossframes and Crown Crossbeam

The existing superstructure dead load was transferred to the new steel-arch supporting system by sequential jacking of the superstructure at each end crossframe and crown crossbeam. The existing pier bearings were removed, and the superstructure was lowered on the new bearings at the end crossframes and crown crossbeam. Jacking was done at one location at a time following the sequence shown in Figure 5.

Before jacking, new bearing stiffeners were connected to the existing plate girders, and permanent steel jacking beams (W33 × 118 at the end crossframes and W27 × 84 at the crown crossbeam) were connected to the stiffeners. All field connections were made with ⅞-in.-diameter high-strength bolts (ASTM A325), and all steel was 50-ksi (344-MPa) high-strength steel. A typical connection detail between an exterior girder and a jacking beam is shown in Figure 9.

Jacks, Deflection Limitations, and Blocking

Sixteen 100-ton jacks, two jacks per lift point, were used to raise the superstructure at the end crossframes (Figure 10) and the hydraulic system shown in Figure 7 was doubled.

The vertical deflection criteria for lifting at the end crossframes were the same as those specified for the end pier jacking. However, the total vertical deflection permitted at the crown crossbeam was reduced to 2.0 in. (51 mm). Because of their close proximity to the end crossframes, the existing intermediate piers served as convenient reference points to check both the total and relative deflections of the girders during jacking.

TABLE 1 END PIER JACKING: CALCULATED AND RECORDED LIFTING LOADS

	Lift Points								
Description	1	2	3	4	5	6	7	8	Total
Calculated	81.4	15.7	40.8	32.1	32.1	40.8	15.7	81.4	340.0
Piers 1 & 4	(362)	(70)	(181)	(143)	(143)	(181)	(70)	(362)	(1512)
Percent	24.0	4.6	12.0	9.4	9.4	12.0	4.6	24.0	100.0
Recorded	110.0	25.4	17.0	60.6	55.2	19.0	24.4	106.0	417.6
Pier 1	(489)	(113)	(76)	(270)	(245)	(85)	(108)	(471)	(1857)
Percent	26.3	6.1	4.1	14.5	13.2	4.5	5.9	25.4	100.0
Recorded	98.4	37.5	39.0	42.0	47.4	40.0	41.0	102.5	447.8
Pier 4	(438)	(167)	(173)	(187)	(211)	(178)	(182)	(456)	(1992)
Percent	22.0	8.4	8.7	9.4	10.6	8.9	9.1	22.9	100.0

Loads are in kips (kN)

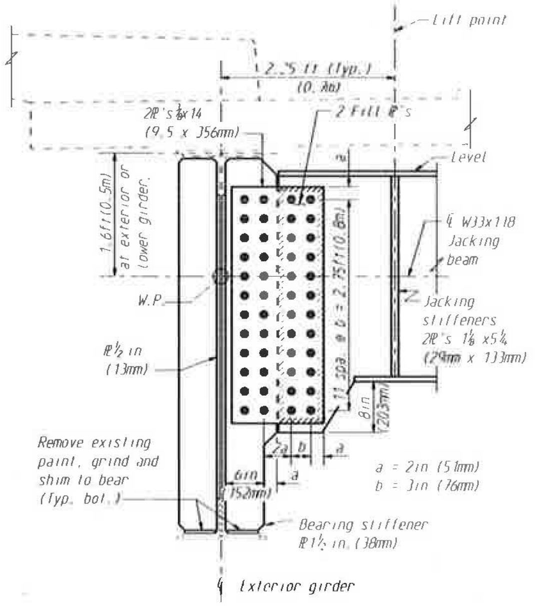

FIGURE 9 Jacking beam to girder connection.

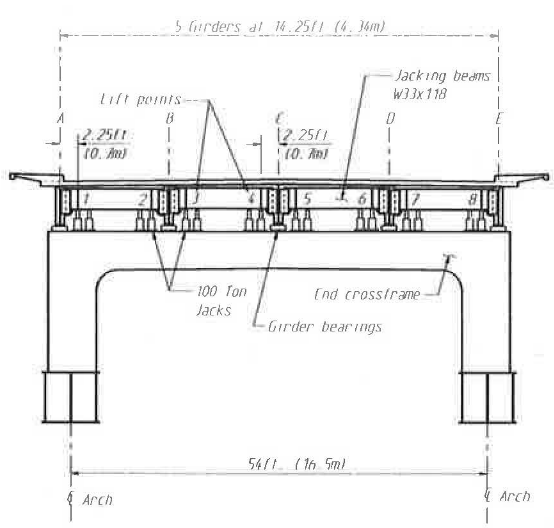

FIGURE 10 End crossframe, jacking beams, and jacks.

During the end crossframe jacking, steel shims were installed as blocking at the existing pier bearings. If a jack should fail, the girders would not be displaced relative to one another by more than the shim thickness of ⅛ in. (5 mm).

Calculated and Recorded Lifting Loads

The existing superstructure had a total dead load of 2,840 kips (12,630 kN). Under final conditions, 2,120 kips (9,430 kN) or 75 percent is transmitted to the arches by the two end crossframes. During jacking, loads greater than these occurred because of the sequence followed.

Computer analyses using STRUDL were performed to determine the magnitude and distribution of the loads and deflections during all jacking sequences. Once the calculated lifting loads were determined, a second analysis was performed to obtain the transverse load distribution to each jack by assuming the total lifting load to be applied as five equal loads at each girder.

Jacking Sequence 3

Figure 11 shows jacking at the first end crossframe. The superstructure was still attached to the opposite existing pier, which prevented the superstructure from moving both longitudinally and transversely. The spaces between the girders and unjacked end crossframe were tightly blocked; this blocking induced a compressive force in the unjacked end crossframe as the arch deflected upwards.

The total calculated load which would lift the superstructure off existing Pier 2 and maintain the original grade was 1,164 kips (5,177 kN). The calculated deflection, which is the sum of the end crossframe deflection, arch deflection, and the upward deflection of the superstructure, varied from 1.3 in. (33 mm) at the exterior girders to 1.6 in. (41 mm) at the middle girder.

The total lifting load, recorded from the jack gauges, was 1,422 kips (6,325 kN), and the measured deflection at liftoff varied from 1.375 in. (35 mm) to 1.625 in. (41 mm). The total recorded lifting load was 22 percent greater than that calculated. The measured deflections were 0.075 in. (2 mm) greater than that calculated at the exterior girders and 0.025 in. (0.6 mm) greater at the middle girder. This provided a good check on the computer analyses.

Jacking Sequence 4

Figure 12 shows jacking at the opposite end crossframe, where the total calculated load that would lift the superstructure off existing Pier 3 was 1,190 kips (5,293 kN). The actual lifting

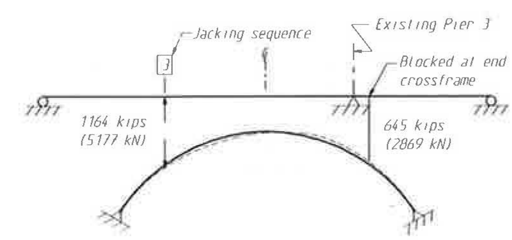

FIGURE 11 Jacking Sequence 3—end crossframe jacking.

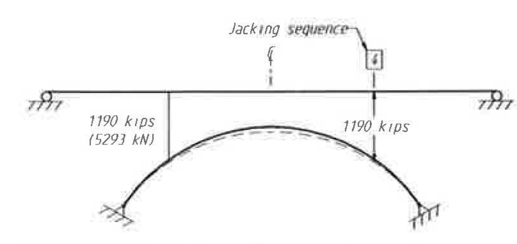

FIGURE 12 Jacking Sequence 4—end crossframe jacking.

load was 1,494.6 kips (6,648 kN) or 26 percent greater than that calculated. The recorded and calculated loads at each jack are presented in Table 2 for Jacking Sequences 3 and 4. Also presented are the percentages of the total at each jack, which exhibit good correlation in predicting the transverse distribution of the total load to the jacks.

Jacking Sequence 5

After completion of the end crossframe jacking, the superstructure at the crown crossbeam was jacked and the bearings installed (Figures 13 and 14). The intent was to induce precompression in each bearing to offset any live load uplift. The total desired precompression was 200 kips (890 kN). The total load recorded was 233.6 kips (1,039 kN) or 17 percent over that desired. A comparison between the recorded and calculated loads at the jacks and the percent of the total at each jack are presented in Table 3.

Final Dead Load Distribution

After completion of the superstructure jacking, the existing deck was resurfaced with a latex concrete wearing course 1½-in. (38-mm) thick. The additional dead load, applied to the superstructure uniformly, was 1.12 kips/ft (16.3 kN/m). The final superstructure dead load distribution to the arches is shown in Figure 15.

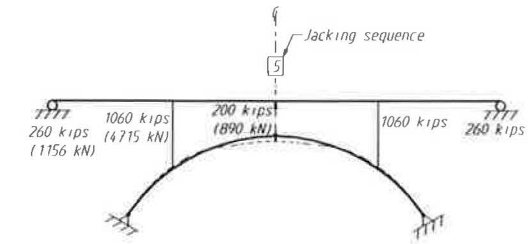

FIGURE 13 Jacking Sequence 5—crown crossbeam jacking.

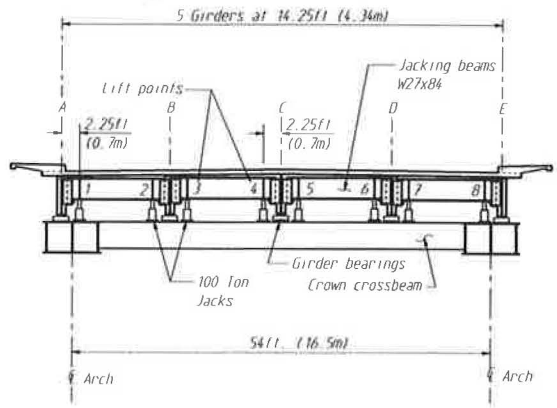

FIGURE 14 Crown crossbeam, jacking beams, and jacks.

Reasons for Higher Lifting Loads

The total loads recorded during the jacking operations were higher than those calculated by 23 percent at Pier 1, 32 percent at Pier 4, and 22 to 26 percent at the end crossframes. There are several possible reasons for these discrepancies:

1. The concrete deck may be thicker than that shown in the original contract plans, which would increase the dead load. The actual dead load is probably within 5 to 6 percent of the calculated dead load on the basis of the calculated and measured deflections.

2. The jacks may not be plumb or loaded concentrically, which may cause the piston to bind or seize up. These internal forces must be overcome to lift the superstructure. They may have caused the jack gauges to record higher lifting loads than those calculated.

3. Frozen bearings and possible composite action of the concrete deck may have contributed to higher lifting loads.

4. The jacks may not have been placed at the same locations that were used for calculating the jack loads. This difference

TABLE 2 END CROSSFRAME JACKING: CALCULATED AND RECORDED LIFTING LOADS

	Lift Points								
Description	1	2	3	4	5	6	7	8	Total
Calculated	273.5	89.6	103.6	115.2	115.2	103.6	89.6	273.5	1164.0
X-Frame 2	(1216)	(399)	(461)	(512)	(512)	(461)	(399)	(1216)	(5176)
Percent	23.5	7.7	8.9	9.9	9.9	8.9	7.7	23.5	100.0
Recorded	348.6	95.6	103.6	165.4	155.6	104.4	101.6	347.2	1422.0
X-Frame 2	(1551)	(425)	(461)	(736)	(692)	(464)	(452)	(1544)	(6325)
Percent	24.5	6.7	7.3	11.6	11.0	7.4	7.1	24.4	100.0
Calculated	279.7	91.6	105.9	117.8	117.8	105.9	91.6	279.7	1190.0
X-Frame 3	(1244)	(407)	(471)	(524)	(524)	(471)	(407)	(1244)	(5292)
Percent	23.5	7.7	8.9	9.9	9.9	8.9	7.7	23.5	100.0
Recorded	352.6	120.6	124.4	142.8	138.2	114.4	119.4	382.2	1494.6
X-Frame 3	(1568)	(536)	(553)	(635)	(615)	(509)	(531)	(1700)	(6647)
Percent	23.6	8.1	8.3	9.5	9.2	7.7	8.0	25.6	100.0

Loads are in kips (kN)

TABLE 3 CROWN CROSSBEAM JACKING: CALCULATED AND RECORDED LIFTING LOADS

Description	Lift Points 1	2	3	4	5	6	7	8	Total
Calculated	46.0 (205)	18.1 (80)	16.1 (72)	19.8 (88)	19.8 (88)	16.1 (72)	18.1 (80)	46.0 (205)	200.0 (890)
Percent	23.0	9.0	8.0	10.0	10.0	8.0	9.0	23.0	100.0
Recorded	37.8 (168)	29.0 (129)	38.4 (171)	29.4 (131)	30.6 (136)	14.6 (65)	10.0 (44)	43.8 (195)	233.6 (1039)
Percent	16.2	12.4	16.4	12.6	13.1	6.3	4.3	18.7	100.0

Loads are in kips (kN)

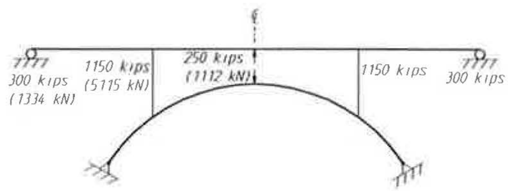

FIGURE 15 Final superstructure dead load distribution.

would cause discrepancies in the transverse distribution of the total lifting loads.

RECOMMENDATIONS FOR LIFTING BRIDGE SUPERSTRUCTURES

Recommendations based on experience at the Washington State Department of Transportation, on guidelines from Orr (*8*), and on the New York State Department of Transportation Standard on Structural Lifting Operations (*9*) are as follows:

1. Calculate the dead load to be lifted. Include the weight of any construction equipment.
2. Locate the lift points and indicate the calculated lifting loads in the contract plans. These should also be shown in the working drawings prepared by the contractor.
3. If jacks are to be used, the cylinder capacity should be from 1½ (*9*) to 2 (*8*) times the calculated lifting load. The manufacturers' name plate and rated capacity should be attached to each jack. The schematic hydraulic layout, including gauges, valves, manifolds, and other equipment, should be shown in the working drawings.
4. If special lift or supporting equipment such as jacking beams or frames are required, they should be properly detailed with particular attention to connection details. Include the type and grade of all materials in the contract and working drawings.
5. So as not to overstress the existing structural members during lifting, indicate the maximum distance to be jacked at each lift point and the maximum relative displacements permitted between adjacent lift points and between adjacent girders. Include any special lift instructions or stage jacking requirements for high lifts.
6. Do not permit traffic on the bridge or the presence of any unnecessary construction personnel near the bridge during lifting. Shim and block as the superstructure is being lifted. In the event of a jack failure, there will be no significant differential settlement and a back-up jack can be quickly installed. Permit traffic on the bridge only after the superstructure has been blocked or shimmed and the load is released from the jacks.
7. Ensure that the superstructure does not move or sway in any direction by establishing maximum permissible deflections and by providing positive restraining systems. This is particularly important for bridges on a steep grade or those located in wind-prone locations.
8. Disconnect any utilities, railings, and traffic barrier cover plates to facilitate lifting.
9. The working drawings, jacking procedures, and calculations should be prepared, stamped, and signed by a professional engineer licensed in the state where the lifting is to take place. This engineer or his designated representative should inspect all aspects of the lifting operation and be present during the lifting.

SUMMARY

1. Two case studies are presented that demonstrate the use of jacks to lift steel bridge superstructures in Washington State.
2. The first case study involves raising the bridge superstructure by as much as 20.6 ft (6.3 m) so that avalanches could pass below without hitting the superstructure.
3. In the second case study, jacking is used as a means to transfer the superstructure dead load from one existing substructure to another. Data were recorded, and a comparison between calculated and measured lifting loads and deflections was presented.
4. Computer analyses are essential in determining the anticipated lifting loads and deflections during jacking. The second case study indicated that the total recorded lifting loads exceeded the calculated loads by 22 to 32 percent.
5. Possible reasons for discrepancies between recorded and calculated lifting loads were given.
6. Recommendations for lifting bridge superstructures were presented.
7. Jacks should be sized for a minimum of 1½ times the calculated lifting loads.

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The superstructure lift for the Capitol Boulevard Bridge was designed by the author, checked by Y. A. Mhatre and H. L. Coffman, and supervised by U. C. Vasishth, G. T. Markich, and C. S. Gloyd, Office of Bridge and Structures, WSDOT. The field data were provided by Kevin Dayton, District 3, WSDOT.

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