Sample Chapter

Failed Bridges
Case Studies, Causes and Consequences
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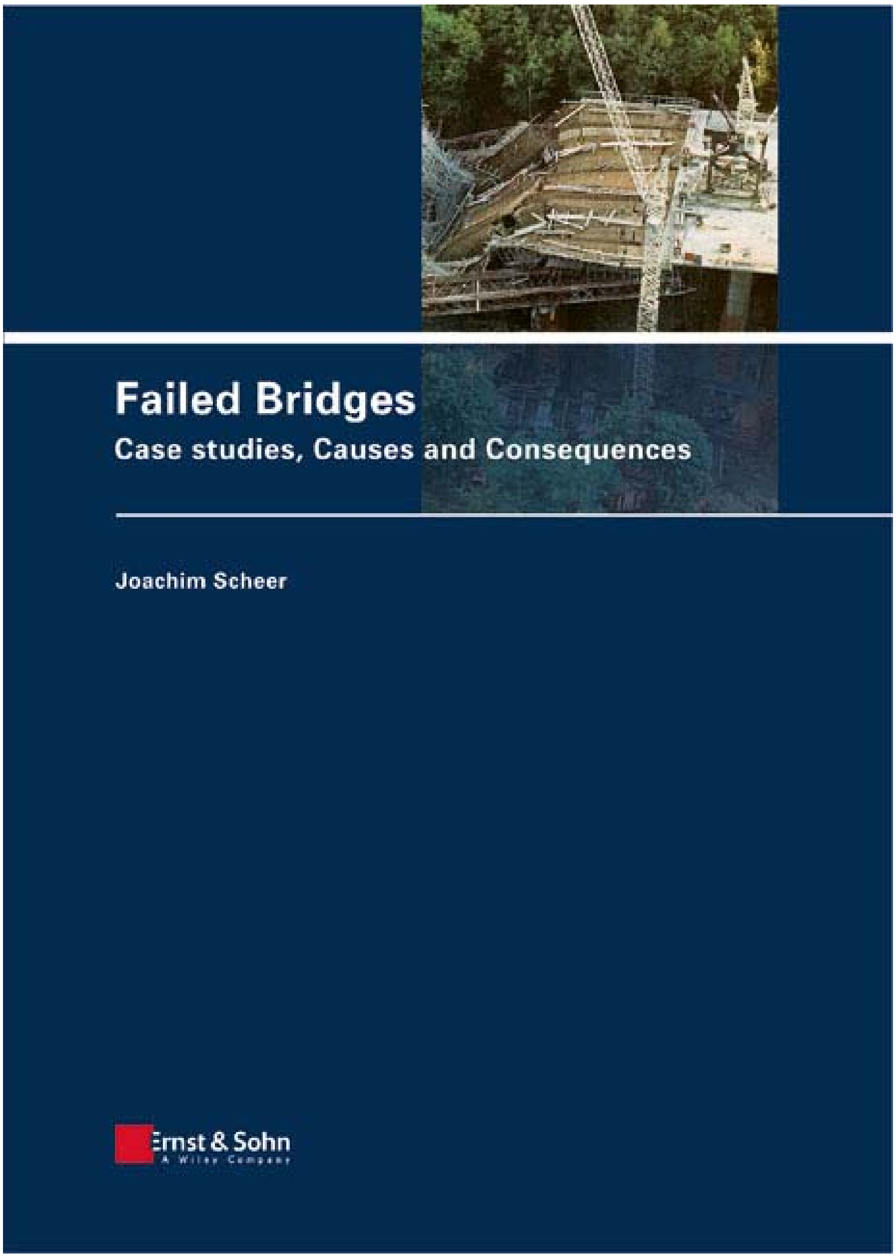

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Table 11 (Continued)

Case No.	Year	Bridge/Scaffolding			Failure	Dead/ Injured	Collapse of scaffold	Length/span (m)	Sources, Fig.
		Location/Type	Country	over					
	1972	Bridge nr. Sacramento, California, 2-span box girder	USA	road	Collapse during striking of scaffolding.				Civ. Eng. 1973 Oct.,74
	1972 .	Road underpass in Ventura	USA	road	Collapse of scaffolding during erection.				Civ.Eng. ASCE, 1973, Oct. 74/5
	1972	Bridge in Victoria	Australia	Loddon River	Support scaffolding collapsed during concreting.	3 D			Report of the collapse of falsework. HMSO, London 1972
	1980	Bridge nr. Sacramento, California, 5-span box girder	USA	American River	Support shoring failed 5 days after concreting of the 67 m middle span.	0	partial	317/67/67	ENR 1980, 14.08., 13
	1981	Road bridge in Bremen	Germany		Scaffolding collapsed during concreting, probably error in structural analysis.	3 I			Newspaper report
	1982	Rhine Bridge nr. Höchst, Vorarlberg	Austria	Rhine	Collapse of timber scaffolding during concreting.	2 D			Newspaper report

11.2 Failure due to inadequate lateral stiffness

Three types of bracing are needed:

– of standards, to ensure the assumed effective length,
– for compressed flanges, to prevent lateral displacement (overturning of truss or solid-web beams) and
– for spindle areas to assure safe transfer of forces normal to the spindle axis.

The examples are intended to show what can happen when this bracing is inadequate – and when it has been omitted entirely.

11.2.1 Inadequate ensuring of the assumed effective length of supports

One example is the scaffolding for a bridge ramp at Cologne-Wahn airport, 1995, Case 11.45. The report [103] states that a number of errors contributed to the accident besides the lack of adequate bracing for the steel tubes butted together as props. The last approx. 50 m long section of the exit ramp was under construction when a serious accident occurred (Fig. 11.1). Steel shoring, consisting of swivelling head plates on spindles, extension tubes, standards, base plates and steel scaffold fittings, had been erected to support the formwork for the elevated reinforced concrete T-beam slab . … Concreting work had got to the stage of the pouring and compacting of the wet concrete in the last third of the ramp when suddenly and without warning a 30 m section of the scaffolding collapsed. Not only was there a huge 750 t pile of concrete, steel and timber, but the heaters installed to produce warm air during the winter also toppled over, the heating oil leaked out and started a fire. Members of the concreting gang slid with the mass of concrete from a height of 10 m. Some workmen succeeded in

Fig. 11.1 Collapse of supporting scaffolding for a bridge ramp at Cologne-Wahn airport. 1995, Case 11.45

jumping from the collapsing scaffolding but one man was buried by the fresh concrete and was killed.

The inquiry was unable to explain the cause of the accident conclusively but the following deficiencies were found:

– General type approval certificates could not be produced for all scaffolding components.
– The scaffolding work was extremely defective, for example short tubes had often been joined together many times to form standards.
– Bracing connections were completely missing at some points. The upper horizontal bracing had often been joined to the thread of the spindles instead of to the extension tube, compensating the difference of diameter with wooden wedges.
– In some cases the scaffold couplings were not even closed.
– Due to a setting out error, the scaffolding had first been constructed too high and had to be corrected. This had been achieved using makeshift supports and wedges.
– Pin-ended columns were frequently not axially loaded.
– Some standards had simply been placed on the ground without base plates.

This example, and many others, shows what incredible carelessness and bungling can occur in the erection of scaffolding – this case could also have been assigned to Section 11.5.

11.2.2 Inadequate lateral bracing of compressed upper flanges of temporary beams

The scaffolding for the bridge over the River Leubas near Kempten in 1974, Case 11.24 is a typical example that has been frequently mentioned in engineering literature and in lectures. As I visited the site immediately after the collapse (Fig. 11.2), I can report from my interviews with those responsible how extrapolation from experience without due consideration of the conditions particular to the case in question can lead to fatal mistakes.

The bridge had a relatively high transverse cross-fall of α = approx. 4 %. If the slab formwork had been placed on scaffolding with vertical temporary beams (Fig. 11.3a, left), wedges would have been necessary at each junction between beams and squared timbers - over 1000 wedges for the entire bridge. The temporary trusses would only have been stressed by the weight load (G) along their main axes (z). To reduce the work of the shuttering carpenters it was decided, as on other occasions for scaffolding with a shorter span but also using rolled-

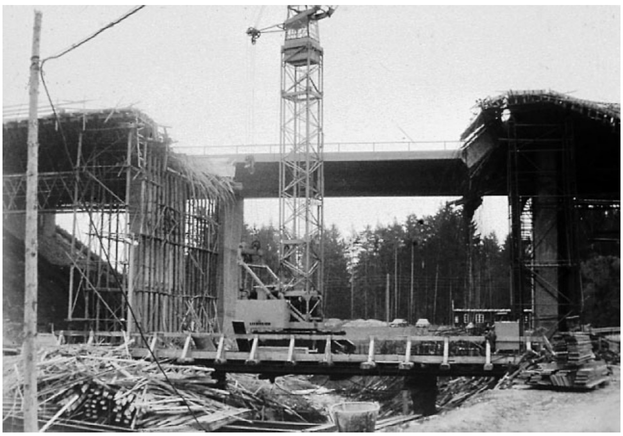
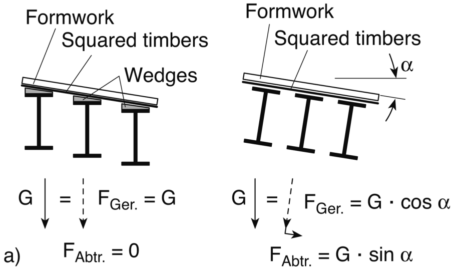


Fig. 11.2 Collapse of supporting scaffolding for a road bridge over the River Leubas near Kempten. 1974, Case 11.24



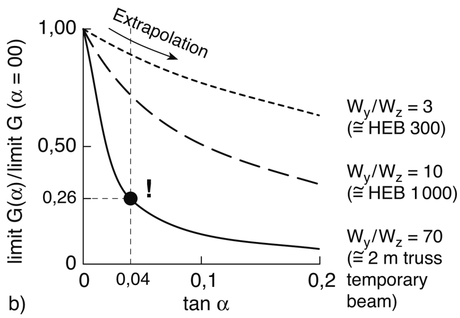


Fig. 11.3 Collapse of scaffolding for a road bridge over the River Leubas near Kempten. 1974, Case 11.24
a) 2 possible scaffolding details for bridges with a crossfall
b) limitation of the load-bearing capacity G(α) by angle α, related to G(α=0)

steel beams, to slope the beams with the same cross-fall as the slab formwork (Fig. 11.3a, right). The horizontal force component resulting from the weight of beams, formwork, reinforcement and concrete is therefore $F_{Abtr.} = G \cdot \sin\alpha$ at right angles to the axis of the web. The scaffolding designers failed, however, to assign this to the wind bracing = stabilization bracing. They rather assumed, without checking, as an extrapolation from previous scaffolding experience that this component would be absorbed by transverse bending in the beams without serious consequences for the structural safety of the scaffolding system. This past experience had been with smaller transverse gradients and rolled-steel beams with a relatively low ratio of the two resistance moments W_y/W_z = approx. 3. Transverse bending had only had a relatively small influence on structural safety (Fig. 11.3b). There were two important differences in the scaffolding for the Leubas bridge:

– the highly “stretched“, approx. 2 m high truss beams had a much higher ratio of section modulus: W_y/W_z = approx. 70 (W_y of the upper chord) and
– the cross-fall α of approx. 4 % was extremely high.

Fig. 11.3b makes these correlations clear. It shows 3 W_y/W_z ratios against the tangent of the angle of inclination α to the limit load *limit G(α)* for the inclined temporary beam to the limit value *limit G(α =0)* for the vertical beam.

Other cases of compressed webs of temporary beams without or with weak transverse bracing are:

– scaffolding for a bridge in Wunstorf. 1979, Case 11.30.
  The accident happened while scaffolding was being removed because the lateral bracing of a 24 m span of several trussed temporary beams under the bridge had been released prior to lifting them out by crane. The beams were not yet attached to the crane when they tipped and fell onto the railway below;
– scaffolding made of beams and timber supports for a flyover in Elwood, 1982, Case 11.34.
  The accident occurred because temporary beams buckled and failed due to a lack of lateral bracing.

11.2.3 Inadequate bracing in the area of screw jack spindles

Two examples are discussed.

– The failure of scaffolding for the Wallenhorst railway overpass. 1966, Case 11.9

The drawing in Fig. 11.4 is taken from [5, p. 25]. It shows the scaffolding built of steel IPB beams on timber props. Between the towers standing on the foundations of the bridge piers and the abutment, there was an arrangement of single-walled intermediate trestles supported on timber piles. IPB 200 beams were located above and below the spindles on the baseplates. There was no bracing over a height of 1.3 m, nor across the longitudinal axis of the bridge. Some of the wooden piles projected more than 1 m out of the ground and were not braced. The collapse occurred in the middle of the main span which was bridged with IPB 800 temporary beams and held only by single-walled intermediate trestles.

The report [5] cites a large number of further deficits: reckless intermediate timber constructions between longitudinal and frame beams; the frame beams were not infilled under the longitudinal beams although concentrated loads of up to 300 kN were introduced; there was nothing to prevent the longitudinal beams from tipping; bracings had not been fitted although required in the drawings from the supervisory engineer and the connections of the round timber members of the bracings were faulty.

As in Case 11.45 described in Section 11.2.1, this example, together with many others, shows how recklessly and irresponsibly some scaffolding erectors go about their work. This case could also have been assigned to Section 11.5 (see also Figs. 11.1 to 11.5).

– Partial collapse of a railway bridge on the Solingen-Ohligs crosslink during sideways jacking. 1977, Case 11.29

An approx. 25 m long bridge for a single-track railway overpass (the cross section is shown in Fig. 11.5a) was being constructed using a 2-span support scaffold – called the concreting scaffold – consisting of steel beams on 3 two-wall scaffolding towers adjacent to the old bridge, which was due for demolition. Fig. 11.6b shows the three towers after the partial collapse. The steel beams of the concreting scaffold were supported at the bridge ends on beams positioned on the scaffolding supports in the axes A′ and C′ and on axis B in the middle support.

A section of this scaffolding – known as the jacking scaffold – was to be used for sideways jacking. This construction is shown in Fig. 11.5c. The superstructure had been partially pretensioned on the scaffolding and therefore had the load-bearing capacity of a single-span beam during jacking.

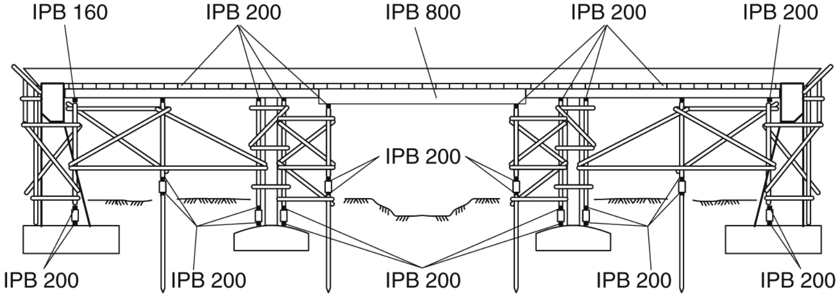


Fig. 11.4 Elevation of scaffolding for the Wallenhorst railway overbridge. 1966, Case 11.9

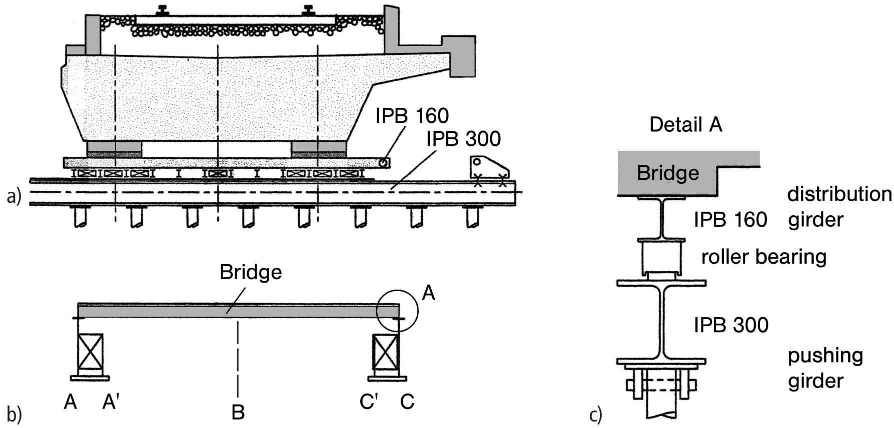


Fig. 11.5 Partial collapse of a railway bridge between Solingen and Ohligs during sideways jacking. 1977, Case 11.29
a) cross-section of bridge
b) scaffolding system
c) jacking construction

After striking the formwork, the superstructure was to lie on the IPB 160 distribution beam in axes A and C and be pushed into position on cylindrical roller bearings along the IPB 300 jacking beam (Fig. 11.5). The jacking beam lay on the head jacks of the scaffolding supports in axes A and C.

The baseplate spindles in axes A′ and C′ were lowered by a few centimeters to remove the formwork. This caused the scaffolding supports at the ends of the bridge to incline resulting in horizontal forces, which finally could no longer be resisted by the scaffolding towers and the jacking construction. The compressed diagonals of the bracing buckled, which increased the stress on the diagonal tension members causing them to slip in their couplings. At the same time the webs of the distribution and jacking beams became plastic.

Fig. 11.6 shows the site after the collapse: (11.6b) gives a general view while Fig. 11.6a shows the north side and 11.6c the south. It can be seen that the distribution beams have fallen out and the superstructure is resting on the jacking beams.

– The collapse of the shoring for a bridge on the Eschwege bypass (1970, Case 11.12) was another failure occurrence with basically the same cause. Here again construction members were not present to transfer the horizontal forces in the area of the baseplate spindles – which were probably extended further than was permissible.

Case 11.33, which is discussed in Section 11.3, was possibly also caused by instability in the area of the head spindles of shoring towers.

a)
b)
c)

Fig. 11.6 Scene of partial collapse of a railway bridge between Solingen and Ohligs during sideways jacking. 1977, Case 11.29
a) view of the jacking scaffold from north
b) overall view
c) view of jacking scaffold from south

11.2.4 A special case

The collapse of the supporting scaffolding for a 9-span express highway in Sweden, Case 11.48, was a result of the very unusual type of scaffolding employed. The bridge consisted of 2 adjacent single-cell prestressed concrete box beams with 30 m spans. Each beam carried 3 traffic lanes. The scaffolding for these beams consisted of 2 truss beams, one behind the other: one for concreting, the other for formwork and reinforcement (Fig. 11.7a). Each of the two truss beams was built up from 2 pairs of standard trusses coupled together. The back ends of these trusses in the formwork beam – in relation to the direction in which the bridge was being constructed – were wide enough apart to allow the sides of the front end of the following concreting beam to pass. The back end of the formwork beam was attached to the upper chord of the concreting beam inside this overlap and in such a way that it remained suspended from the concrete when the concreting beam was removed. The concreting beam could then be used again as a formwork beam.

The concreting beam was suspended from the cantilever arm of the previously concreted section and supported on scaffolding towers next to the bridge columns. It extended into the next span to approximately the quarter point.

The scaffolding failed shortly after concreting had reached the support cross section. It began with lateral deformation of the bottom chords of one of the two truss beam pairs in the cantilever section (Fig. 11.7b) followed by the entire cantilever section of the concreting beam, which then caused the collapse of the formwork beam. According to reports, the entire collapse lasted about 15 minutes.

The structural analysis had assumed that the two bottom chords of each pair were held together at the tip of the cantilever by a horizontal gusset plate. But this gusset plate had

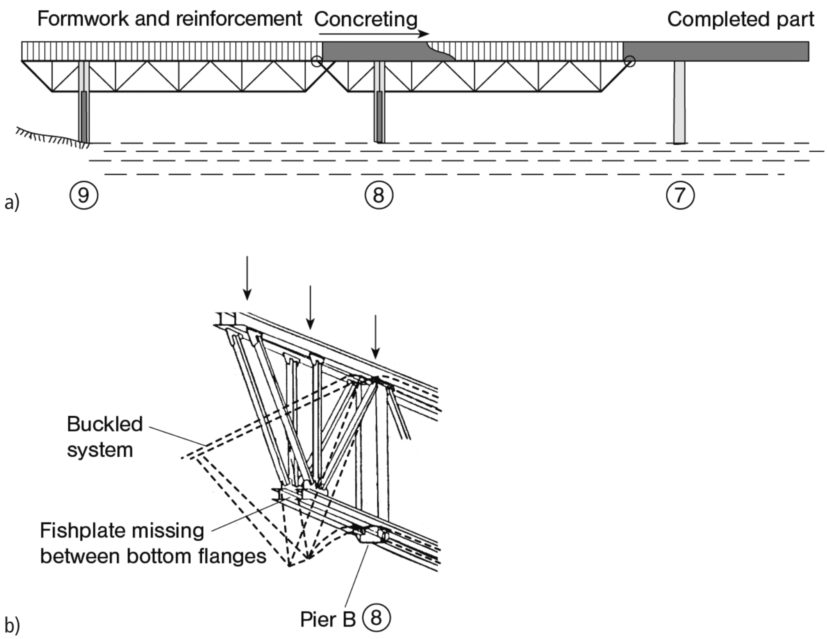


Fig. 11.7 Scaffolding for an express highway in Sweden, Case 11.48
a) scaffolding system
b) failure

been forgotten in the drawings. The truss became unstable in the area of the 1st bottom chord node with 3 compression members and 1 tension member in each truss wall and buckled.

The case could also have been assigned to the category "Coordination errors in design or between design and construction".

11.3 Failure due to poor foundations

The 3 failure occurrences due to poor foundations first mentioned in the overview in Section 11.1 have similar causes. In each case settling of auxiliary foundations led to

- redistribution of forces acting on the supports with a "hard" foundation on the piers of the old bridge causing overstressing and collapse (scaffolding for the Limburg motorway bridge. 1961, Case 11.7, Fig. 11.8a);
- unequal settlement of column feet due to different loads and locally different subsoil conditions, in this case fill (Case 11.8, bridge in Ludwigshafen, 1966) and to overstressing of scaffolding parts;
- misalignment of supports due to excessive soil compaction leading to unscheduled horizontal forces which, together with other causes resulted in a collapse (scaffolding of the Lösegraben bridge in Lüneburg. 1967, Case 11.11, Fig. 11.8b).

The real cause of the disastrous collapse of scaffolding in East Chicago in 1982 (Case 11.33) was probably quite different. It is worth taking a closer look because of the various mistakes made in the construction of the scaffolding.

The ramp needed to connect a ground level road with an express freeway required a multi-span bridge first curved in the horizontal plane and then running in a virtually straight line (Fig. 11.9a). It was built as a prestressed concrete single cell box beam (see Fig. 11.9b for dimensions) with spans of up to approx. 55 m. According to reports the longitudinal gradient was up to 3.6 % with – as can be seen in Fig. 11.9b – a transverse gradient of up to approx. 3 %.

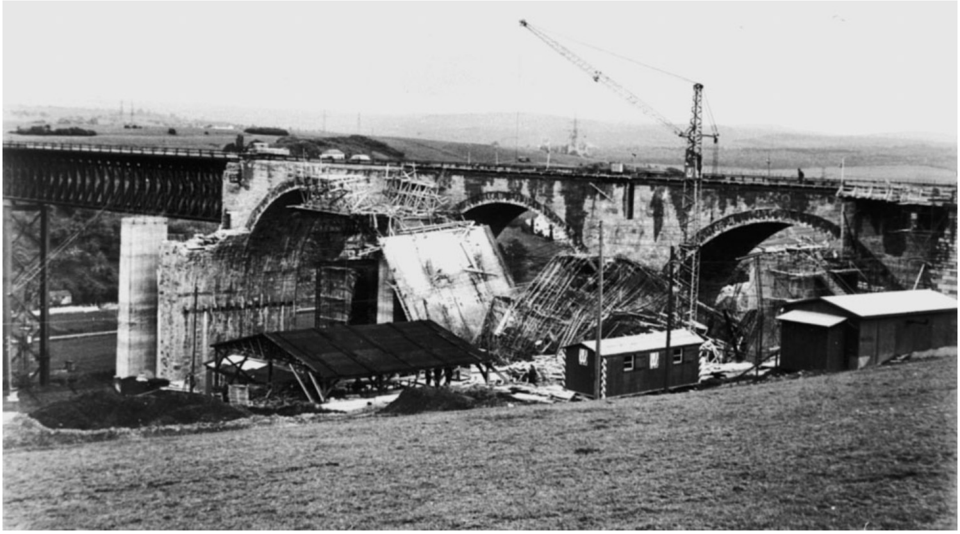
Conventional scaffolding was used with heavy-duty load towers at the piers and the third points of the spans and 915 mm deep longitudinal I-beams with spans of up to 18 m. These lay on 610 mm deep transverse I-beams themselves supported on the top spindles of the towers.

The foundations of the support towers at the third points were precast concrete slabs, 1.5 m x 1.5 m. Contrary to the design, these were 300 mm instead of 530 mm thick. 30 cm thick squared timbers were to be placed between the concrete slabs and the baseplates of the support towers for load distribution (Fig. 11.9b).

The ground beneath the slabs had neither been analyzed nor prepared for scaffolding although the top 1.5 to 2.7 m consisted of a fill of ashes, cinders and sand "with an oily smell". In some areas compressible black organic mud had also been found. Soil compaction, according to the plan, must have been around 155 kN/m^2.

The superstructure was built in sections between the fourth points of the spans in two phases. First the trough consisting of a floor slab and webs was concreted, followed by the traffic deck.

The cantilever bridge section over support 407 was completed and prestressed. The trough and the transverse bearing beam over support 408 had been concreted for the section projecting over support 408 (Fig. 11.9a). The trough was already in place for the further sections up to the end of the bridge for connection 410 to the freeway. Work was in progress to concrete the road deck from the end of the cantilever arm in front of support 407 in the direction of 408 – upwards due to the longitudinal incline – when the support tower 407.2 collapsed. The fall of the trough and the road deck, which had not yet hardened, from a height of approx.


a)


b)

Fig. 11.8 Scaffolding collapses caused by settling of temporary foundations
a) Limburg motorway bridge. 1961, Case 11.7
b) bridge in Lüneburg. 1967, Case 11.11

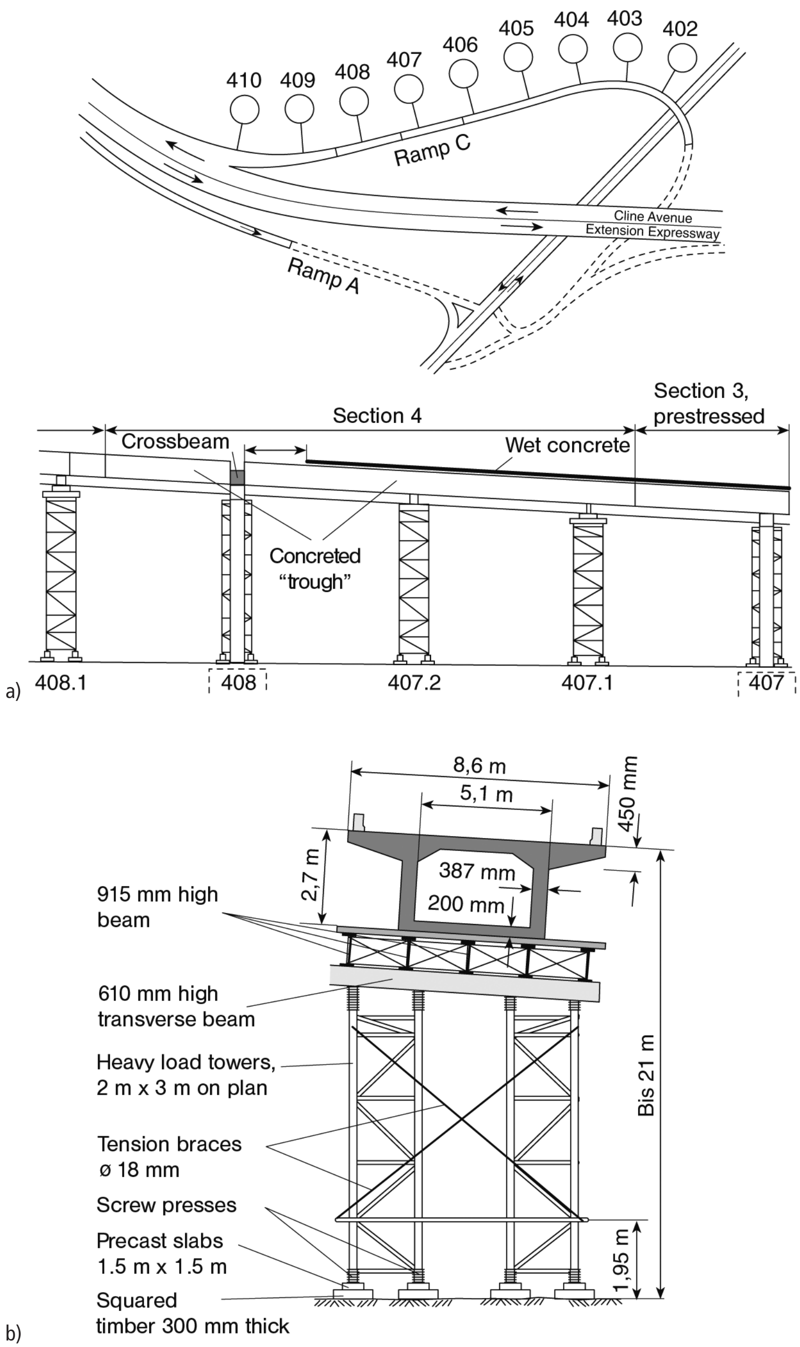


Fig. 11.9 Scaffolding collapse during construction of a bridge in East Chicago. 1982, Case 11.33
a) system
b) cross section of shoring tower and box girder