Precast Stress Ribbon Pedestrian Bridges in Czechoslovakia

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Pedestrian bridges (or footbridges as they are called in many parts of the world) have been used by man since antiquity to cross rivers, deep gorges and narrow mountain passes. One commonly used method has been to suspend a walkway from a fiber rope catenary span — somewhat similar to a primitive suspension bridge.

The stress ribbon concept borrows the suspension bridge principle but develops it further by using high strength materials and modern engineering technology — especially precasting and prestressing methods.

In a prestressed concrete stress ribbon bridge high strength steel cables are passed through a series of precast concrete components, the deck assembly of which can be tensioned from stiff abutments. Whereas in a suspension bridge the main load carrying component is the cable with the deck acting as a stiffening element, in a stress ribbon bridge both the cable and the deck can be independently tensioned, thus adding considerable rigidity to the structure.

It should be mentioned that in contrast to highway bridges, pedestrian bridges carry relatively light loads. However, the effects (deflection, volume changes and other displacements) due to temperature differentials can be quite significant.

Recently, nine precast stress ribbon pedestrian bridges were designed in Czechoslovakia. Seven of these bridges have now been built and are in full operational use. The structures are aesthetically beautiful and have become well recognized landmarks enjoyed by all segments of the population.

The superstructure of the pedestrian bridge is formed by a prestressed band which is attached to rigid end abutments. The deck is made up of precast concrete segments which are suspended

Describes the design features, construction procedure and economic considerations involved in building several stress ribbon pedestrian bridges in Czechoslovakia using precast concrete components.

on high strength steel bearing cables and then shifted along the cables to the specified position. Prestress is induced in the deck after the joints between the segments have been concreted in place. This compression gives the structure sufficient rigidity to withstand the dead and live loads.

The pedestrian bridges vary from one to four spans. The longest span length is 472 ft (144 m) and the maximum length of the structures is 1329 ft (405 m). A summary of the major characteristics of the bridges designed by the Enterprise Dopravni stavby is given in Table 1. Figs. 1 through 7 present a panoramic view of the stress ribbon bridges built through 1985.

DESIGN FEATURES

The key design features of the pedestrian bridges are summarized in this section. All the bridges have a uniform

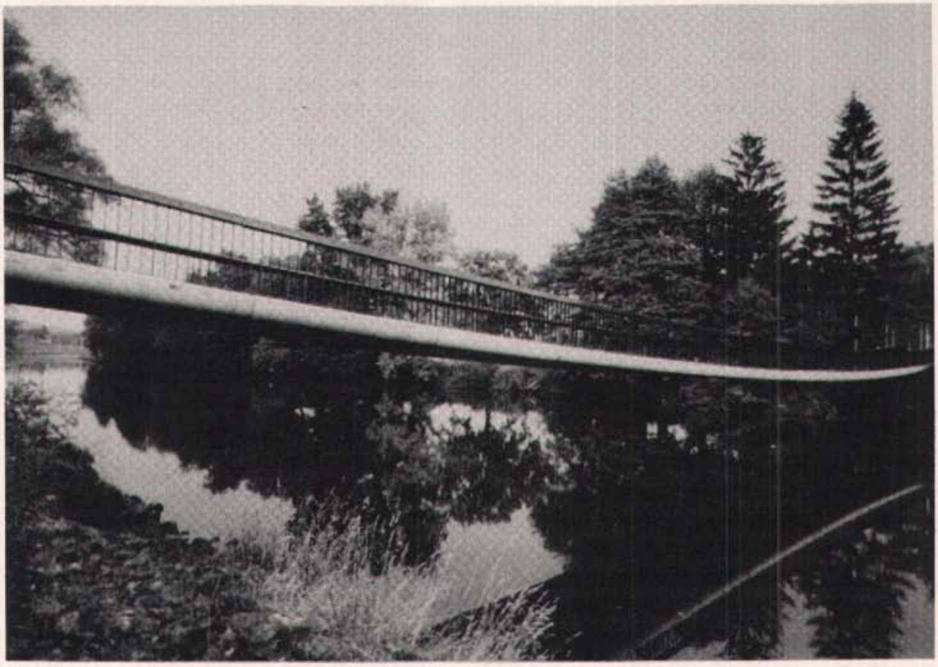


Fig. 1. Pedestrian bridge in Brno-Bystrc.

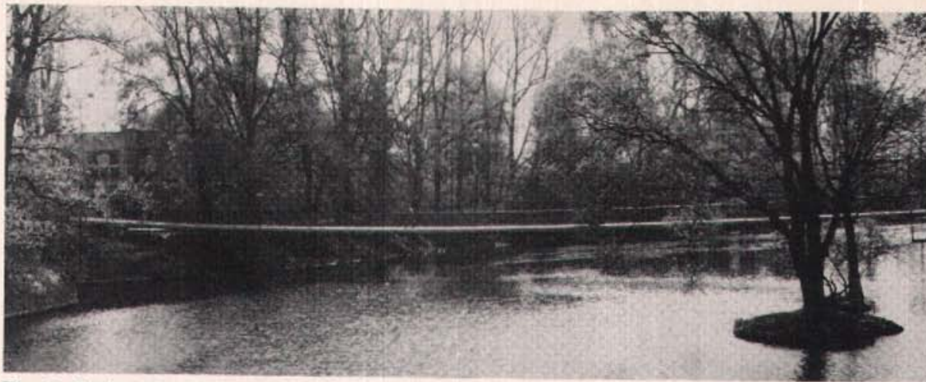


Fig. 2. Pedestrian bridge in Brno-Komin.

Fig. 3. Pedestrian bridge at Kromeriz.

cross section as shown in Fig. 8. The decks are assembled from precast segments 12 ft wide, 10 ft long and 1 ft thick (3.80 x 3.00 x 0.30 m). The clear width between the railings is 10 ft (3.00 m). As part of the design, the segments have two openings of 5 in. (12 cm) diameter for laying trunk and electric cables.

The longitudinal arrangement (see Fig. 9) is determined by the given structural system and the selected horizontal force. With a dead load (g) of 19.9 kips per ft (27 kN/m), the maximum horizontal force H_g may equal 5394 kips (24 MN). The superstructure between single supports has the shape of a catenary which differs only slightly from a second degree parabola. The pedestrian bridges were designed for a live load (p) of 9.68 kips per sq ft (4.00 kN/m²).

The precast segments were manufactured with high strength concrete hav-

Fig. 4. Pedestrian bridge in Prague-Troja.

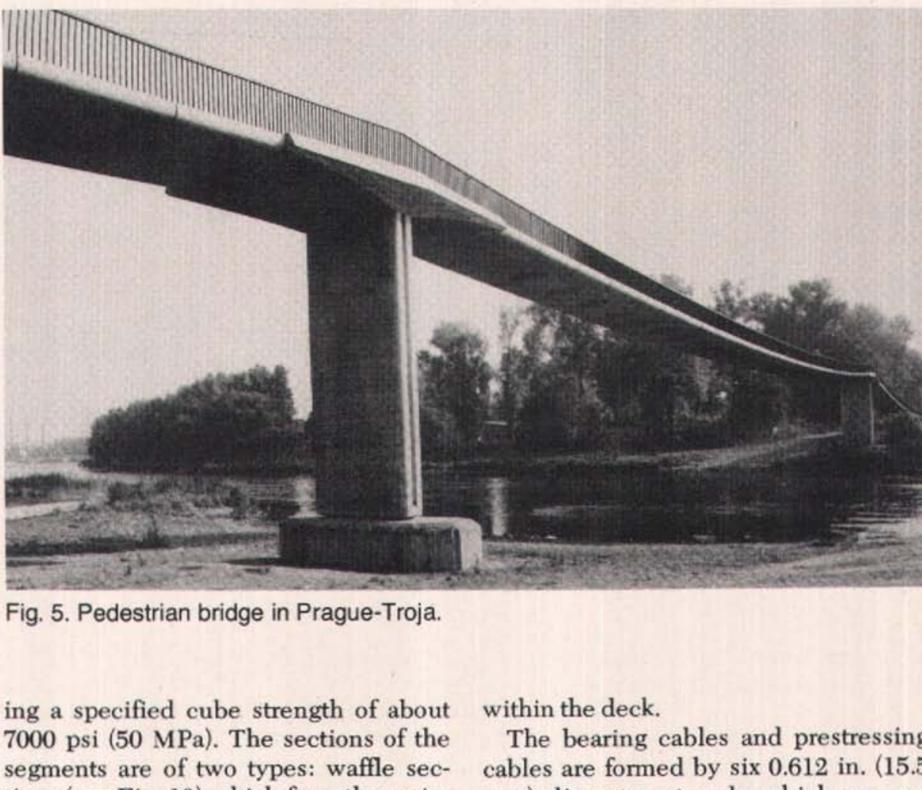


Fig. 5. Pedestrian bridge in Prague-Troja.

ing a specified cube strength of about 7000 psi (50 MPa). The sections of the segments are of two types: waffle sections (see Fig. 10) which form the major part of the span and solid sections which are located at the supports.

During erection the segments are hung on bearing cables which are held in a pair of troughs. After erection the segments are prestressed by the second group of cables placed in the ducts within the deck.

The bearing cables and prestressing cables are formed by six 0.612 in. (15.5 mm) diameter strands which are anchored in pairs. The specified strength of the cables is 261 ksi (1800 MPa).

The bearing cables are arranged in two ways, depending upon their number and magnitude of the horizontal force. When 2 x 6 cables are used, the cables are placed in each trough in one row.

Table 1. Major characteristics of stress ribbon pedestrian bridges in Czechoslovakia.

Pedestrian bridge	Figure number	Number of spans	Maximum span length, l_{max} ft (m)	Sag of maximum span, f ft (m)	Length of structure ft (m)	Year of erection
Brno-Bystrc	1	1	206.69 (63.00)	3.94 (1.20)	226.38 (69.00)	1979
Brno-Komin	2	1	255.91 (78.00)	4.43 (1.35)	275.59 (84.00)	1985
Kromeriz	3	1	206.69 (63.00)	3.94 (1.20)	248.03 (75.60)	1983
Radonice	—	1	206.69 (63.00)	3.94 (1.20)	242.78 (74.00)	1984
Prerov	19	2	221.46 (67.50)	4.69 (1.43)	334.65 (102.00)	1983
Zatec	—	2	247.70 (75.50)	5.25 (1.60)	406.82 (124.00)	Design
Prague-Troja	4, 5, 21	3	314.96 (96.00)	5.54 (1.69)	856.96 (261.20)	1984
Nymburk	6, 7	3	334.65 (102.00)	6.50 (1.98)	758.53 (231.20)	1985
Velke Brezno	—	4	472.00 (144.00)	9.52 (2.90)	1329.40 (405.20)	Design

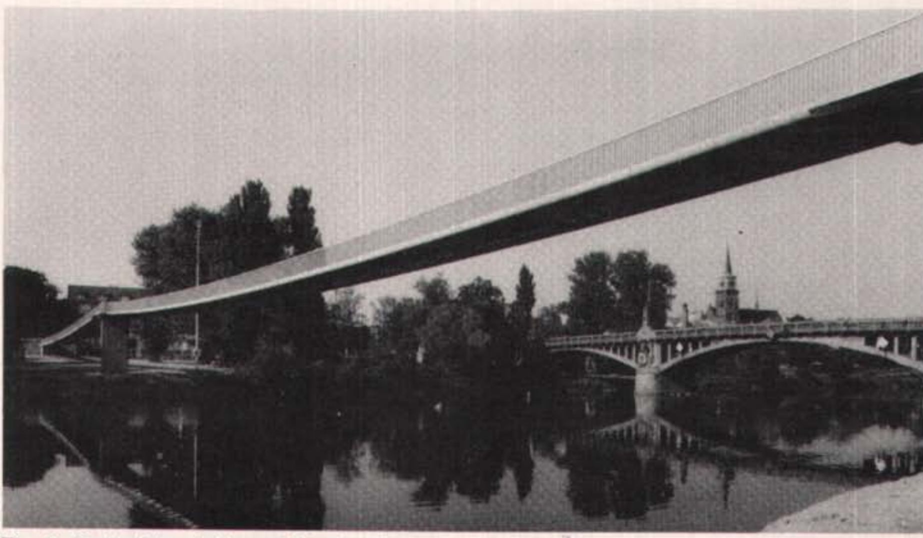


Fig. 6. Pedestrian bridge at Nymburk.

However, when 2 x 12 cables are used, the cables are placed in two rows.

During erection the segments are carried only by one-half of the cables placed in the lower row. After erection the cables placed in the upper row are pulled through and tensioned. The amount of cable tensioning determines the final configuration of the structure.

The bearing cables are protected by concrete with a specified cube strength of about 5800 psi (40 MPa) which is prestressed. The minimum cover of the cables is 2.17 in. (5.5 cm).

The prestressing cables are placed in the segments in ducts formed by concrete. However, in the joints, saddles and anchorage blocks the ducts are formed by steel tubes.

The end segments of all the stress ribbon pedestrian bridges are placed during erection on elastomeric bearing pads situated on abutments (see Fig. 13). This allows the deck to move up and down, respectively, as the temperature falls and rises. At the end abutments there

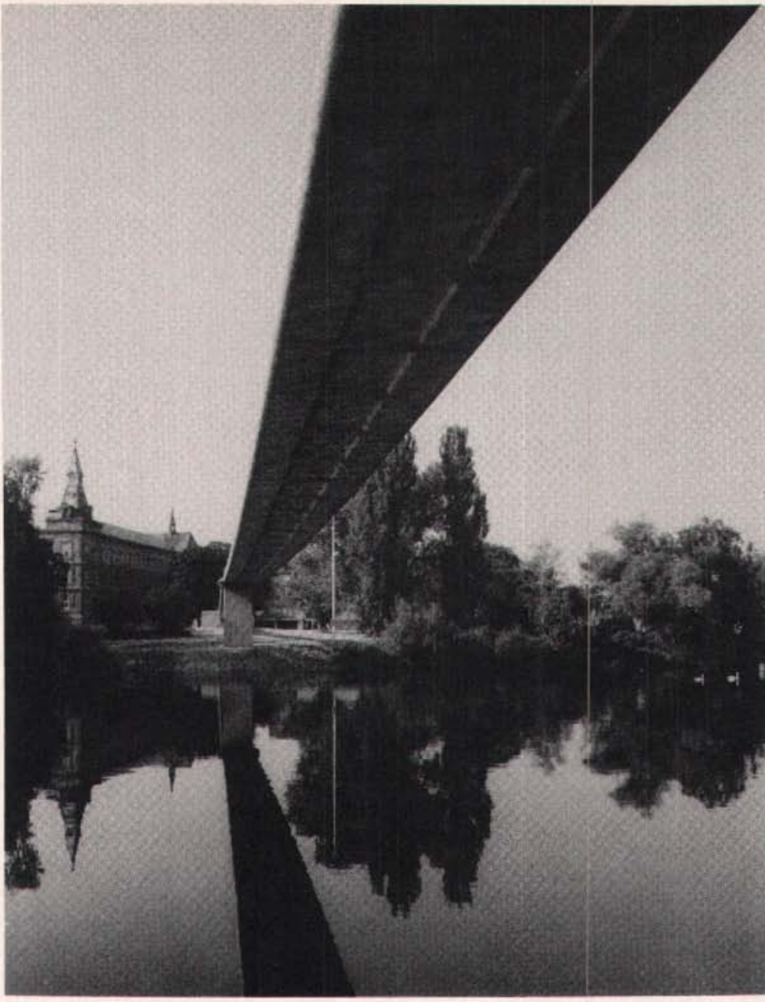


Fig. 7. Pedestrian bridge at Nymburk.

are anchorage blocks which connect the superstructure with prestressing.

It is important to be aware that very large horizontal forces [as high as H = 6745 kips (30 MN)] will need to be transmitted from the end abutments into the foundation. This load transfer can be done by:

(a) Flexurally rigid bored piles.
(b) Raking compression and tension piles.
(c) A combination of wall diaphragms and micropiles.
(d) Soil and rock anchors.

The decision as to which is the most suitable foundation to use will depend largely upon the geological conditions at the site and the available mechanical equipment.

The shape of intermediate supports of stress ribbon bridges with more than one span is shown in Fig. 11. The structural design of the supports resulted from the chosen method of erection. In contrast to a continuous stress ribbon bridge built in Freiburg, Ger-

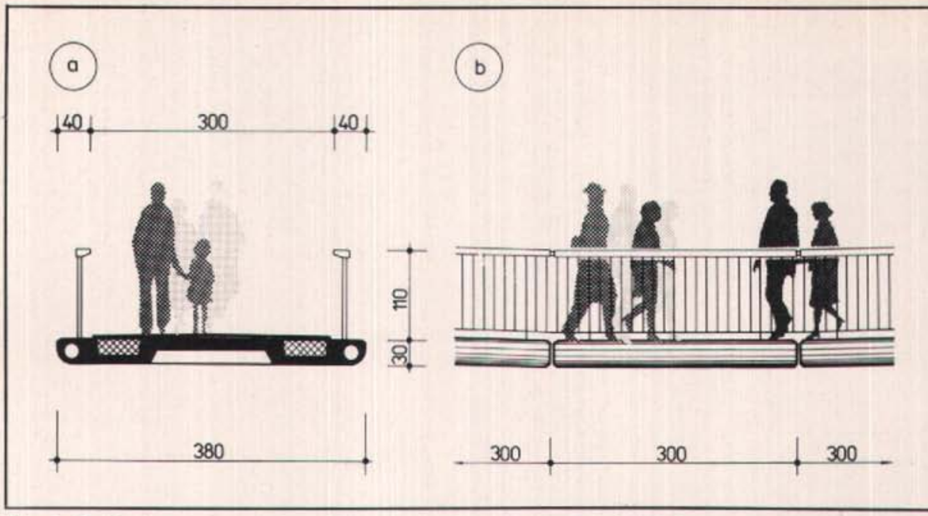


Fig. 8. Typical architectural configuration of pedestrian bridge: (a) cross section; (b) elevation. Note: 1 cm = 0.3937 in.

many,[1,4] in which the concrete band is supported by pendulum piers with tapered tables, the pedestrian bridges described herein use a concrete saddle. The saddles are cast after erecting all the segments in the formwork which is hung on adjacent segments. Instead of pendulum piers having a concrete hinge at the footing, the saddles are frame connected with pendulum piers having reinforcing steel.

In the intermediate supports the bearing cables pass over steel saddles formed by two, 7.9 in. (20 cm) diameter, circular cylinders which are supported by steel footings connected with screws to the monolithic piers. Since the stress of bearing cables gradually increases during their tensioning, at the time of erection of segments and casting the joints, troughs and saddles, their elongation also increases.

Similar to curved ducts of prestressed concrete structures, friction is generated in the steel saddles. The result is that its horizontal component loads the piers with a bending moment. Since the concrete hinge is designed at the footing, the stability of the piers must be ensured. Therefore, the intermediate supports were reinforced with temporary steel struts (see Fig. 17).

The above structural design makes it possible to remove the dependence of the construction of the concrete saddles on the sag of bearing cables which varies in the course of their construction according to temperature and loading. The increased stress is taken by the reinforcing steel.

The pavement is formed by epoxy concrete 0.4 in. (1 cm) thick which at the same time waterproofs the deck. The railing is either made of ropes or thin walled vertical posts. The pedestrian bridges are illuminated at night either by lamps located in the handrails or by reflectors placed on poles outside the structure.

CONSTRUCTION PROCEDURE

The precast segments are manufactured in a steel mold in a one-day manufacturing cycle. Concrete is compacted by surface vibrators. In order to reduce the effects of creep and shrinkage of

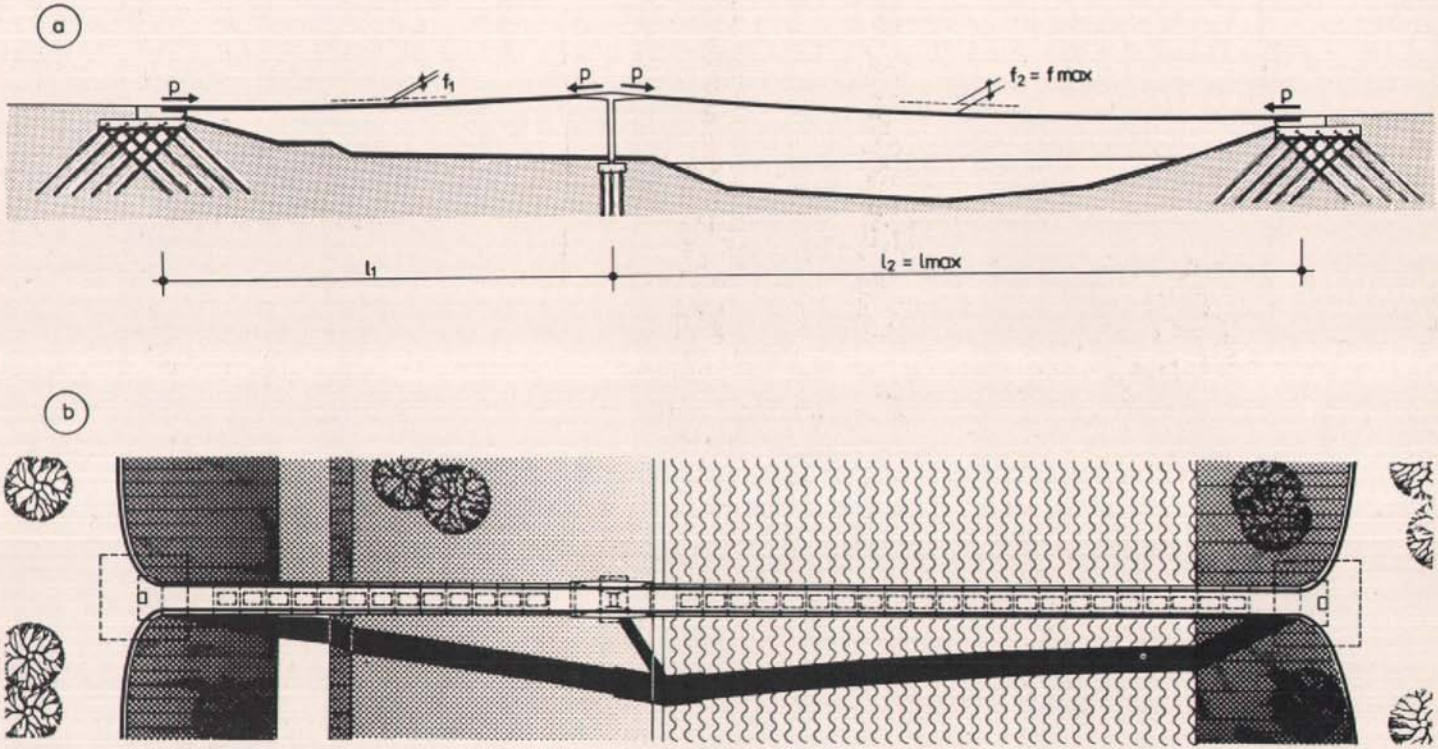


Fig. 9. Typical structural configuration of pedestrian bridge: (a) longitudinal section; (b) plan.

Fig. 10. Typical precast segment on bearing cables.

Fig. 11. Pedestrian bridge showing intermediate support.

concrete, the segments are cast from 6 to 12 months before the erection of the superstructure.

Fig. 12 shows schematically the construction sequence of a two-span pedestrian bridge. Bridges with more than two spans are constructed in a similar way. When foundations, anchorage blocks and intermediate supports are constructed, the assembly of the superstructure is started. This assembly is divided into five stages as follows:

(a) First, the end segments are placed on elastomeric bearing pads on the abutments (see Fig. 13). In the troughs of these precast members are steel members which determine the position of the bearing cables. Then, steel struts are placed on the intermediate supports which secures the stability of the piers (see Fig. 12a).

(b) The bearing cables are drawn by a winch. The strands are wound off from the coils and at the side abutment are slowed down by a cable brake which also ensures the same length of all strands in the cable. Each cable is also attached to an auxiliary rope which enables the back drawing of the hauling rope (see Fig. 12b). After drawing, each

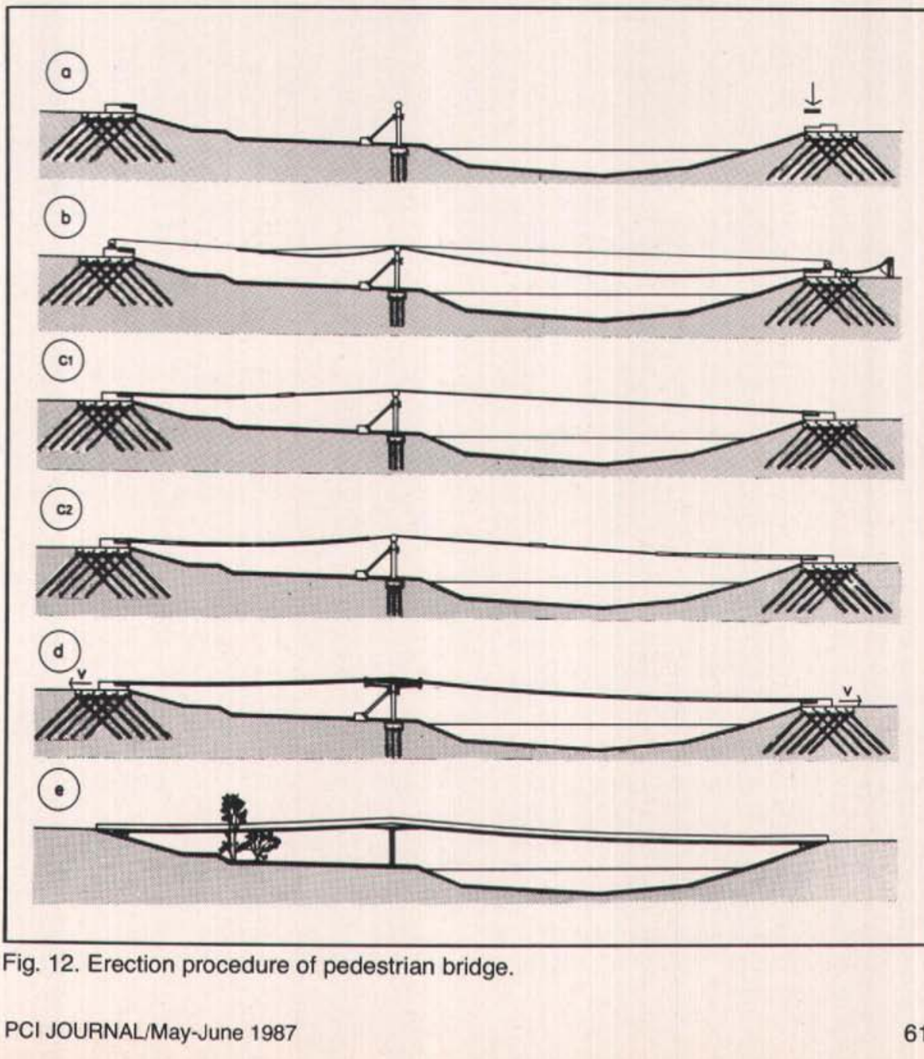


Fig. 12. Erection procedure of pedestrian bridge.

Fig. 13. Erection of the end segment.

cable is tensioned to the prescribed stress. In structures where the cables are placed in one row, all bearing cables are drawn. However, in structures where the cables are placed in two rows, only those cables which are placed in the lower row are drawn.

(c) The segments are erected in single spans by means of a crane truck (see Figs. 14 and 15). The member assembled is first placed under the bearing cables and is lifted as far as the cables touching the bottom of the troughs. Then, hangers are placed in position, secured by screws and the segment is fixed to the hauling rope. The auxiliary cable is attached to the hauling cable to make its back drawing possible. Then, the segment is directly shifted along the bearing cables into the determined position by a winch (see Fig. 16). Here, steel tubes are placed which will form cable ducts in joints. The segment is then drawn to a member assembled earlier. This process is repeated until all the members are assembled (see Figs. 12c1 and 12c2).

(d) When all the segments are assembled, in structures with two rows of bearing cables, the cables in the upper row are pulled through and tensioned. In this manner the design shape of the structure is obtained. Then, the falsework of monolithic saddles is hung on the neighboring segments (see Fig. 17), prestressing cables are pulled through and the reinforcement of the troughs and saddles is concreted at the same time. In order to reduce the effects due to creep, shrinkage, temperature and accidental movement of pedestrians, the deck is partially prestressed as early as

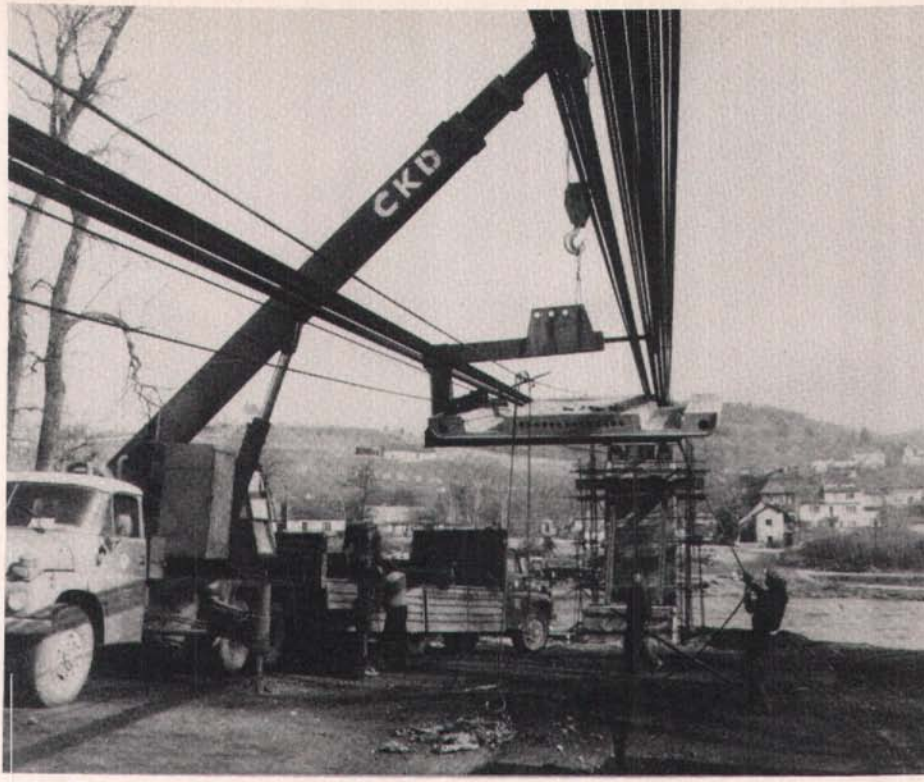


Fig. 14. Erection of a segment of the first span.

possible. When sufficient strength is reached, all the cables are tensioned to a predetermined value.

(e) After grouting the cables, the railings are erected and the pavement is cast. Then, the load test is carried out (see Fig. 12e).

STATIC ANALYSIS

The static action of the pedestrian bridge is determined by the construction method and structural arrangement of its members. The analysis was carried out for two different stages as follows:

(a) Erection stage (see Fig. 18a).

(b) Service stage (see Fig. 18b).

It should be noted that the shape and the state of stress of the structure at the end of the erection stage determine the stress pattern which develops in the structure during its service life.

(a) Erection Stage

During erection the precast segments are hung on the bearing cables which are now loaded by:

- — Their own self weight.
- — The weight of the segments.
- — The weight of the concrete closure joints.
- — The weight of the troughs and saddles.
- — The effects caused by temperature changes.

As a result, the cable acts as a perfectly flexible unit. Since the cables are not connected to the supports with the steel saddles, they can be shifted along them according to the imposed load.

Fig. 15. Erection of a segment of the second span.

Hence, the cables act as a continuous unit which crosses fixed supports.

Now, at a change in load, friction is developed in the steel saddles, the magnitude of which may reach as much as 10 percent of the cable tension force. The behavior of the bearing cable is also affected by the change of its elongation in the anchorage blocks and by possible displacement of the end supports. Therefore, the analysis at the erection stage has taken into consideration all of the above factors.

In the design computations, due account must be taken of the required final shape of the structure after erection has been completed. The various operations such as concreting the joints, the troughs and the saddles were planned in such a way that the effective friction forces at the saddles were almost negligible. As a result, the anchorage stress of the bearing cable was determined with such accuracy that the final erected shape was within 1 in. (2.54 cm) of the design shape.

(b) Service Stage

After concreting the joints, troughs and saddles, the structure distributes all the other loads, i.e., the prestress effect, the weight of the pavement and railings, the live load, the deformation of supports, the temperature changes and the effects due to concrete creep and shrinkage because the thin concrete band is stressed not only by the normal force but also by the shear force and bending moment.

Since the structure is very slender, local shear and bending stresses develop only under point load and at the supports. Because these stresses are relatively small, they do not affect the behavior of the entire structure. This makes it possible to analyze the structure in two closely related steps:

In Step 1, the stress ribbon is analyzed as a perfectly flexible cable which is hinge connected to the supports. The hinges are assumed to occur at the third points of the length of the concrete saddles where their rigidity is already sub-

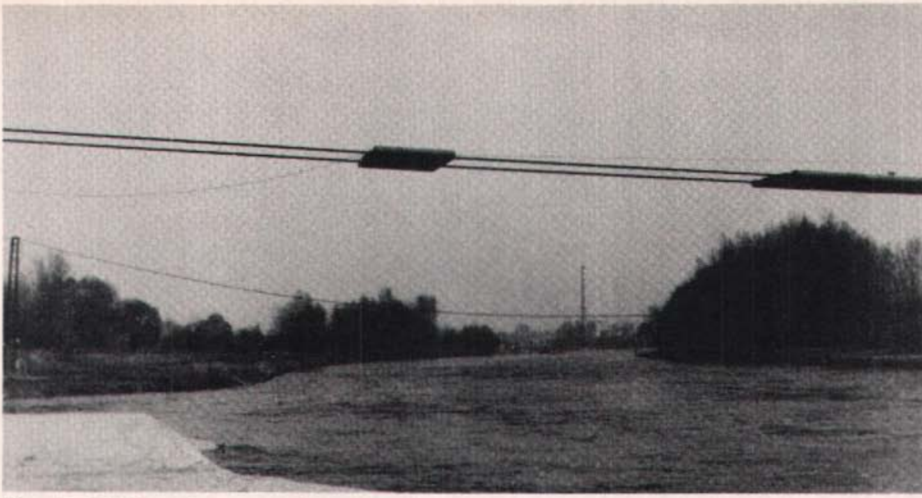
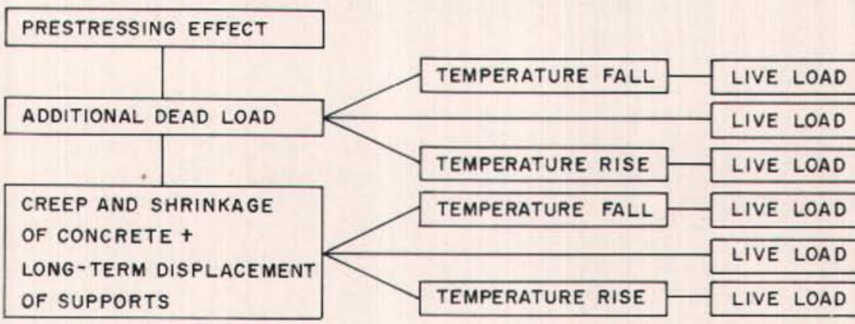


Fig. 16. Shifting of a segment.



Flow chart showing analytical sequence of stress ribbon pedestrian bridge.

stantially high. To facilitate the analysis, standard computer programs for a continuous cable are used. From these programs the unknown horizontal force (H_i) and the deformations of the supports of single spans can be determined.

In Step 2, shear and bending stresses in single spans are evaluated using the analytical method given in Ref. 2 for cable stayed bridges. The stresses in the concrete saddles and piers are obtained by analyzing the intermediate support for loading as if the reaction is acting at the supports of single spans. At the end supports it is assumed that the eccentric hinge connection is developed only under very heavy load.

The analysis of stress ribbon pedestrian bridges is carried out according to the flow chart shown above.

The design assumptions and quality of workmanship were checked by measuring the deformations of the superstructure at the time of prestressing and during the loading tests. Only a few of the key results in a typical structure are given here. Since the shape of a stress ribbon structure is extremely sensitive to temperature change, the temperature profile was carefully recorded at all times.

The strength capacity of the pedes-

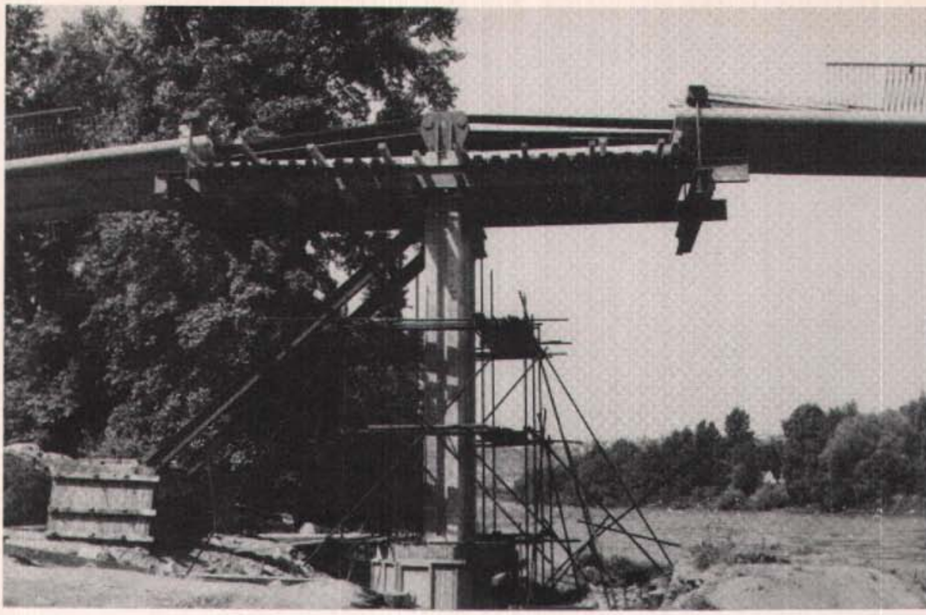


Fig. 17. Falsework of the saddle.

trian bridge at Prerov was tested using three, fully loaded vehicles (see Fig. 19), weighing 10.5, 18.9 and 21.9 tons (11.6, 20.8, and 24.1 t) with the axle load reaching as high as 8.7 tons (9.6 t). In this test both the total bearing capacity of the structure and the bearing capacity during localized bending were evaluated. Fig. 21 shows the measured and calculated deformations of the structure for two vehicle positions.

The results of the load tests on the (currently) longest pedestrian bridge built in Prague-Troja for the first load are shown in Table 2. It should be noted that in this load test, only the deformations at the midspans and horizontal displacements were measured.

The pedestrian bridge was first load tested using 38 heavy trucks weighing from 2.8 to 8.4 tons (3.1 to 9.3 t). The trucks were placed on the entire span length of the structure as shown in Fig. 20. Then, the vehicles were placed on each span. As can be seen, the compatibility of the results is very good.

Table 2. Deflections at midspans of pedestrian bridge in Prague-Troja.

	Span 1 in. (cm)	Span 2 in. (cm)	Span 3 in. (cm)
Calculation	2.24 (5.70)	7.87 (20.00)	1.57 (4.00)
Measurement	2.20 (5.60)	7.32 (18.60)	1.57 (4.00)

DYNAMIC ANALYSIS

Stress ribbon structures are very sensitive to dynamic loads because of their low bending stiffness, low vibration damping and low natural frequencies. For this reason, it was necessary to analyze their dynamic response both theoretically and in the field.

The pedestrian bridges were modelled as systems of cables connected at the tops of pendulum piers. Structures with only one span were solved as an isolated cable with nonflexible sup-

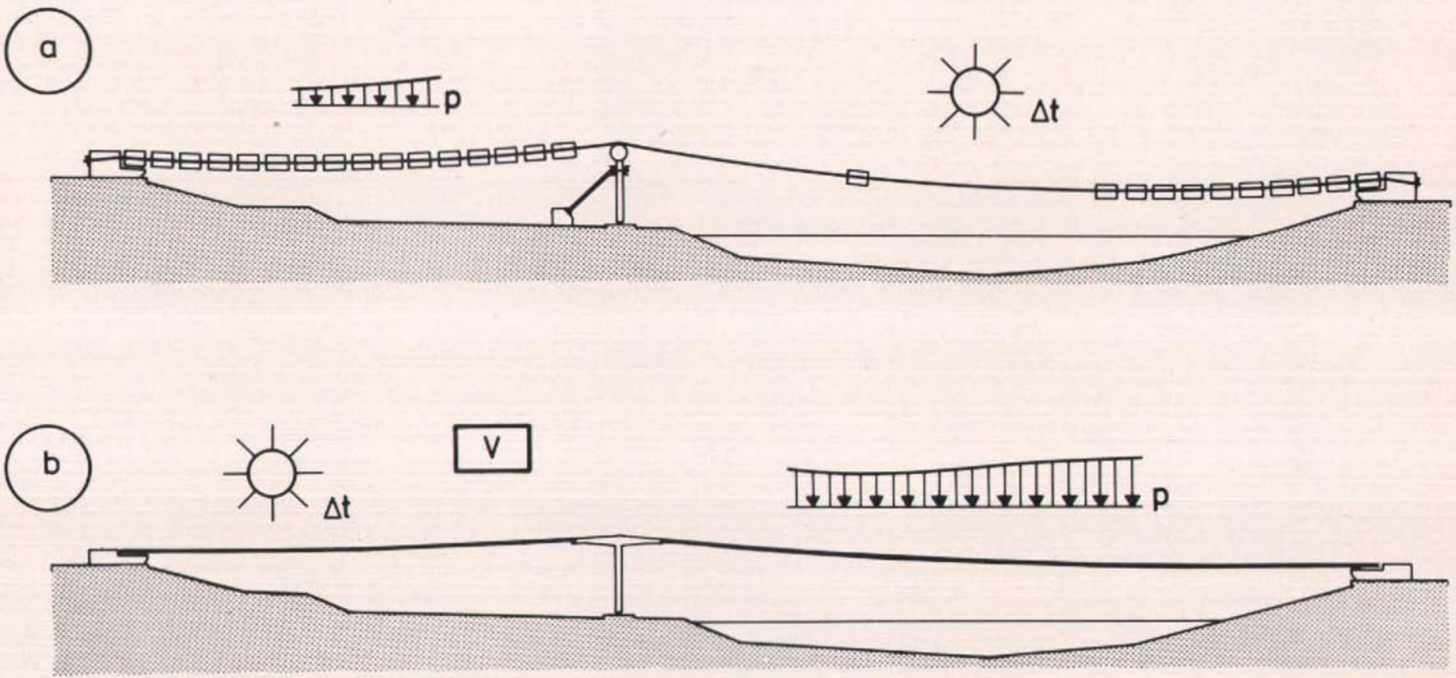


Fig. 18. Static action on pedestrian bridge: (a) erection stage; (b) service stage.

Fig. 19. Load test using three heavy vehicles — Pedestrian bridge at Prerov.

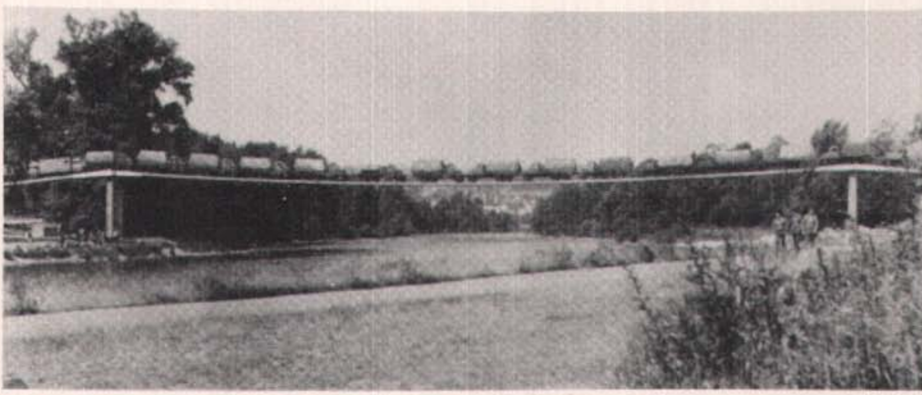


Fig. 20. Load test using 38 vehicles — Pedestrian bridge at Prague-Troja.

ports. In solving forced vibration, the harmonic force passing through the bridge was substituted by a harmonic force acting at midspan. The frequency of the harmonic force was assumed equal to the frequency which is most commonly caused by pedestrians, namely $f = 2$ Hz.

The stress ribbon structures in Brno-Bystrc, Brno-Komín, Prerov and Prague-Troja were subjected to dynamic tests. In the course of the load tests the agreement of excited natural frequencies with theoretical values was investigated. The structures were excited either by a human force, or by a pulse rocket engine, or by a mechanical rotation exciter. For illustration, Fig. 22 shows the theoretical and excited modes of vibration of the pedestrian bridge in Prague-Troja.

In addition, the damping of vibration was investigated. For example, in the case of the Prague-Troja pedestrian bridge, it was demonstrated that in the region of dynamic displacement of the midspan of the second span, the logarithmic decrement of damping ν varied from 0.008 to 0.012.

From the results of the dynamic tests, which are described in greater detail in Ref. 3, the following conclusions can be made:

1. Stress ribbon pedestrian bridges

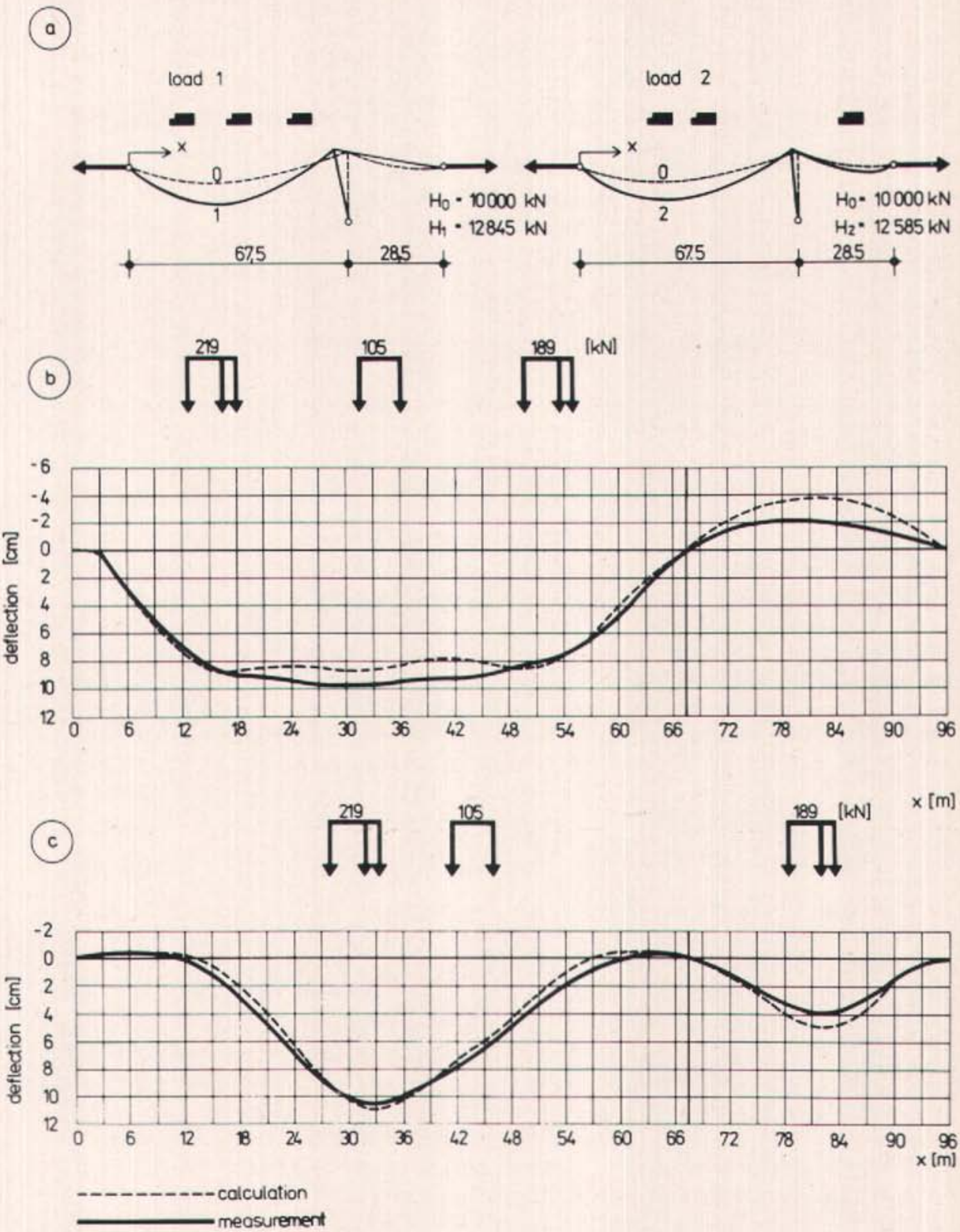


Fig. 21. Load test of pedestrian bridge at Prerov: (a) scheme of structure; (b) deformation due to load 1; (c) deformation due to load 2. Note: 1 cm = 0.3937 in.; 1 m = 3.2802 ft; 1 kN = 0.2248 kip.

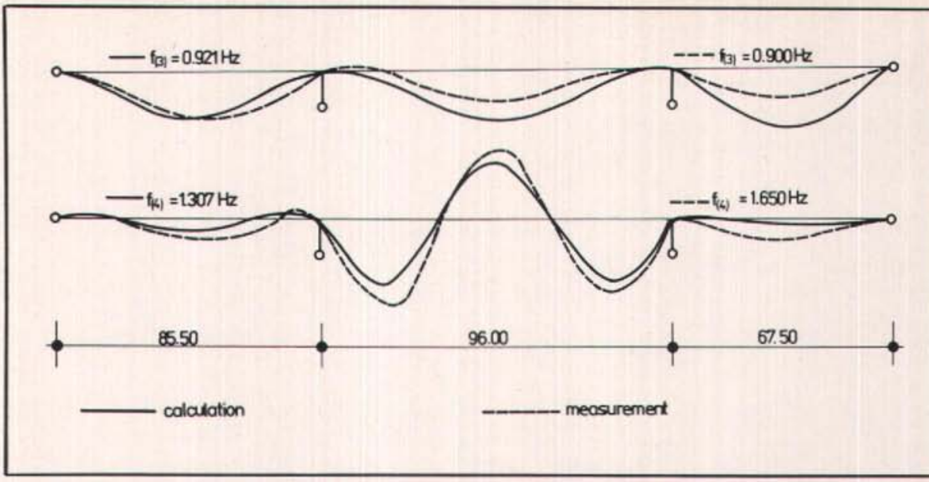


Fig. 22. Theoretical and exciter-induced modes — Pedestrian bridge at Prague-Troja.

cannot be overstressed either by excessive vibration caused by people or by acts of vandalism.

2. The speed of motion caused by people should not exceed the recommended limit of 0.95 in. per second (24 mm/sec) that is recognized as a limit of unpleasant feeling (bouncing effect).

ECONOMIC CONSIDERATIONS

The cost of building stress ribbon pedestrian bridges depends on the span length, the cable sag, the total length of the structure, and especially on the geographic site and existing geological conditions. The general availability of local materials, skilled labor and mechanical equipment might be other factors to consider.

All of the bridges designed by Enterprise Dopravni stavby have been built across rivers, in places where it was not permissible to erect piers in the river bed. Also, for architectural reasons, cable stayed structures were excluded in the planning stage. Since centerings with temporary supports in the river bed would be prohibitively expensive, the cost of stress ribbon structures was compared with cast-in-place cantilever segmental structures.

A comparison of the costs for the above two structural systems showed that the prices were about the same for single span structures. In this case, the prestressed band was chosen because the components could be assembled quickly and easily and the architectural form was aesthetically pleasing.

The economic advantage of stress ribbon bridges is manifested in multispan structures. For the same span and sag, the horizontal force of multispan structures is the same as for single span structures. Also, the cost of the foundation is the same. Therefore, the cost of a pedestrian bridge per square foot (per m^2) decreases as the number of spans increases. For illustrative purposes, two types of structures are compared. These were designed to span the Vltava River in Prague.

The structure, built in 1984 (see Figs. 4, 5, 20 and Table 1), is formed by a prestressed concrete band of three spans [280.5, 314.9 and 221.5 ft (85.5, 96.0 and 67.5 m)] making up a total span of 816.9 ft (249 m).

The pedestrian bridge follows the

Table 3. Consumption of materials for pedestrian bridge in Prague-Troja.

Part of bridge	Materials	Stress ribbon structure	Cantilever structure
Superstructure	Concrete cu ft (m^3)	9182 (260)	22,955 (650)
	Prestressing steel tons (metric tons)	66.9 (68.0)	53.1 (54.0)
	Reinforcing steel tons (metric tons)	40.4 (41.0)	58.1 (59.0)
Substructure	Concrete cu ft (m^3)	13,773 (390)	7240 (205)
	Reinforcing steel tons (metric tons)	26.1 (26.5)	31.5 (32.0)
Foundation	Wall diaphragms sq ft (m^2)	8396 (780)	3444 (320)
	Micropiles quantity (No.)	36	—

contour of the ground on both banks of the river. It rises from the abutments to the intermediate supports and at the midspan bridges the river. The large horizontal force [6016 kips (27 MN)] is transferred into the foundation by a combination of wall diaphragms and micropiles.

The compared structure was formed by a continuous beam of four spans [83.7, 193.6, 314.9 and 218.2 ft (25.5, 59.0, 96.0 and 66.5 m)]. The span length of the structure was the same, i.e., 816.9 ft (249 m).

The superstructure of the I cross section had a depth varying from 13.12 to 3.94 ft (4.0 to 1.2 m). The bank spans were concreted in place on centering and the central span was built by the cantilever method. The bridge was founded on wall diaphragms.

Table 4. Cost of pedestrian bridge in Prague-Troja (in million crowns).*

Part of bridge	Stress ribbon structure	Cantilever structure
Superstructure	2.236	6.605
Substructure	0.920	0.995
Foundation	4.661	1.020
Total	7.817	8.620

* Because of the different monetary systems used in Czechoslovakia and the United States, it is not possible to give a meaningful conversion rate from Czech crowns to U.S. dollars.

Tables 3 and 4 present the quantities

of materials and the costs of both the compared structures. It is apparent from the tables that even with the high price of the foundation, the total cost of the stress ribbon bridge is lower than the cost of the cast-in-place cantilever structure.

CONCLUSIONS

Through 1985, the National Enterprise Dopravni stavby has constructed about 3000 linear ft (900 m) of precast stress ribbon bridges. Currently, other such bridges are in the design and planning stages.

In retrospect, it can be stated confidently that stress bridges are:

— Easy to assemble and quick to erect
— Functional
— Aesthetically beautiful
— Economical
— Enjoyed by the people

The above advantages far outweigh the cost of transmitting a large horizontal force. In the author's bridge department, designs have been developed which further extend the capabilities, range of application and economy of stress ribbon bridges. It is his belief that such structures will find wide application in the future.

ACKNOWLEDGMENT

The design system of the pedestrian bridges described in this paper were developed by the National Enterprise Dopravni stavby in Brno, Czechoslovakia. The dynamic analyses and tests were carried out by Dr. Pirner from Tazus (the Institute for Testing of Structures) in Prague.

The design of these pedestrian bridges was awarded the first prize in the Czechoslovakian competition for young engineers and architects.

The structural design, static and dynamic analyses of the above bridges are described in detail in Ref. 3. If any reader is interested in securing this publication, please contact the author:

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NOTE: Discussion of this paper is invited. Please submit your comments to PCI Headquarters by February 1, 1988.

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