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Study of Cable-Stayed Bridges for Pipelines

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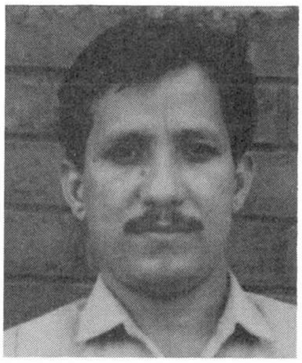

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Summary

Cable-stayed pipeline bridges have very high vertical stiffness but due to their small width and the probability of coinciding there resonant frequency with the peak of horizontal gust spectra, they are vulnerable to wind gust loading. In this work a special emphasis has been laid to investigate the effect of horizontal stiffening system arrangements using cable and cable trusses of different configurations along with aerodynamic and seismic analysis.

1. Introduction:

Cable Stayed pipeline bridges (CSP-Bridges) are generally narrow needing additional stiffening in the horizontal plane. They are vulnerable to wind gust loading due to their small width and the fact that their resonant frequencies coincide with peaks in the horizontal gust spectra. As in any cable stayed bridge, it has inclined stays emanating from one or more points in the pylons and holding the deck of the bridge at intermediate locations between the main supports, thus imparting a high degree of vertical stiffness to the bridge.Since a few studies (5,6,7,8,9,10,11) are only available on the CSP- bridges the designer has little guidelines available for selecting the geometry of the bridge and the sectional properties of its elements. keeping in view the size of the pipeline and access required for repair and maintenance gangs, the width of the bridge has been considered between 2.5m and 7.5m, yielding main span to width ratio in the range of 25 to 35.

2. Bridge Types

Two types of bridges have been taken for the study as shown in Fig. 1.
Type-A: Single span with towers at the ends, as used in the hills and,
Type-B: Three span with towers in between,as commonly used in the plains.
In Type-B Bridges two cases have been studied:

i. Without any horizontal "offsets" at the towers for supporting the stiffening cables and

ii. with an "offset" projecting horizontally at right angles on both sides of the bridge axis at the tower locations and supporting the stiffening cables.

A bridge span range of 50m to 400m total span (main span 50m to 200m for Type-A and 55m to 220m for Type-B) has been considered, which is close to the reported economical span range of 90m to 270m for cable stayed bridges (8,9).

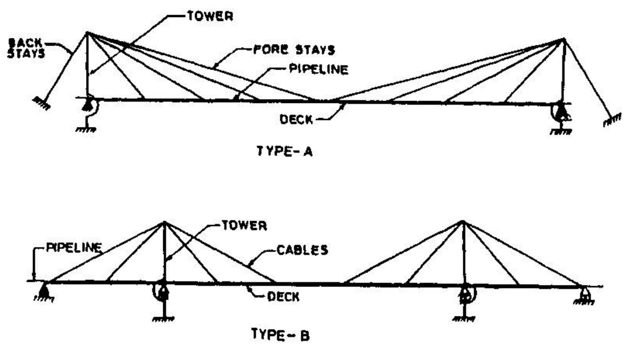

FIG. 1 ELEVATION OF CSP- BRIDGES

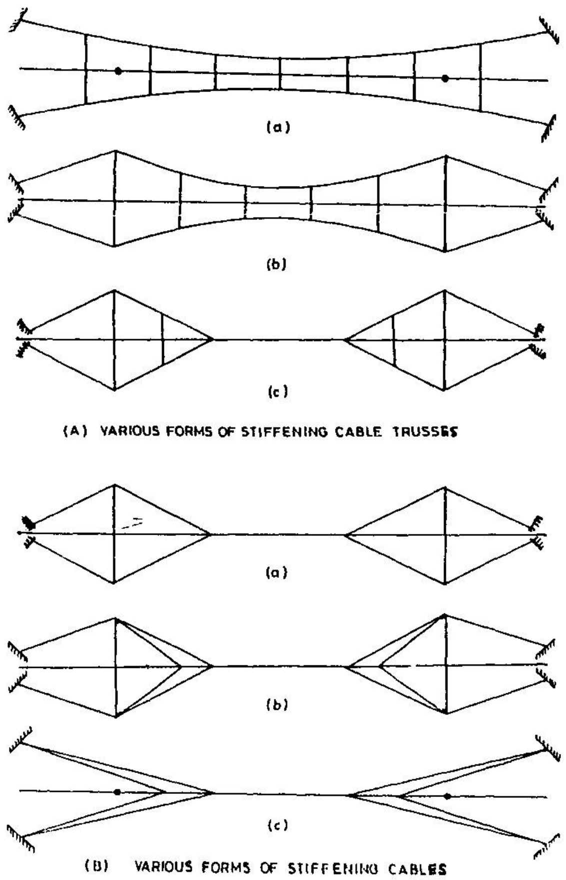

FIG. 2 VARIOUS HORIZONTAL STIFFENING ARRANGEMENT

3. Lateral Stiffening Arrangements

Special emphasis in this work has been laid to systematically investigate the effect of horizontal stiffening arrangements as shown in Fig.2 needed to stabilize the bridge against lateral loads since the narrow bridges have a high degree of susceptibility to wind/earthquake oscillations. The stiffening arrangements studied are cables and cable-trusses of different configurations in the horizontal plane. Firstly,the most effective location for the cable connection with the deck was investigated and the same has been used for the various cases. The criteria adopted for the design of the deck for the lateral stiffness is that its deflection under the lateral loads should not exceed about 1/180 of the span.

4. Static and Dynamic Analysis

Static analysis for various loading cases has been carried out by

(1) considering the structure to be linearly elastic, and

(2) including the effect of cable nonlinearity.

The stiffness matrix method has been used for both static and dynamic analysis. Dynamic analysis study includes the dynamic behavior of CSP-Bridges under wind as well as earthquake loads. To carry out the dynamic analysis the mode shapes and natural frequencies of vibration have been determined by three dimensional free vibration analysis which uses the inverse iteration technique coupled with Strum sequence property of the characteristic polynomials of the eigen value problem (1). In all, 15 modes of vibration have been considered in the analysis and the dynamic response is computed for a specified base motion. However, the dominant modes are only 2 to 5. The maximum seismic displacement responses (SRSS-values) for the six bridges have been evaluated using the average response spectra specified in the IS: code (IS:1893-1984) for 2% damping.

The wind loading on the CSP-bridges is. considered in two parts; the static wind loads due to the steady component of wind and the fluctuating wind loads due to the horizontal gustiness of the wind. The response to the fluctuating load is determined using statistical concepts of stationary time series. Pertinent data on CSP-bridges is evaluated (modes and natural frequencies) to estimate the displacement responses. The vertical vibrations (cross-wind) caused by the action of vertical component of wind turbulence would be insignificant due to the use of highly perforated decks in pipeline bridges coupled with the large vertical stiffness available in the bridge system.

The dynamic analysis for wind loads has been carried out using Davenport's formulation (2,3,4). Some modifications, however had to be made for the cable stayed bridges. The method was also tested for an example illustrated by Davenport (2). Both the bridge configurations could be analyzed using the same procedure.

5. Results of the Analysis

The main findings of the study are as follows:

(a) Static vertical load analysis:

For the linear static analysis the maximum deflection/span ratio in Type-A bridges is found under vertical loading for 50m, 100m and 200m bridges as 1/335, 1/397.5 and 1/230 respectively; whereas; when the nonlinearity of cables is considered the deflection to span ratios are 1/334, 1/397 and 1/229 respectively. It is thus seen that there is little cable non-linearity effect. This is on account of the small LL/DL ratio.

For the linear static analysis the maximum deflection/span ratio in Type-B bridges is found under the vertical loading as 1/328.2, 1/462.0 and 1/229.65 for 100m, 200m, and 400m span bridges respectively. When the nonlinearity of cables is considered the deflection to span ratios are 1/327.6, 1/460 and 1/229 respectively. The nonlinear analysis takes 2 to 5 iterations for convergence. The cable non-linearity in Type - B bridges is found in significant.

(b) Mean wind load analysis:

The maximum lateral deflection to main span ratio under the basic wind speed of 44 m/sec for Type-A and Type-B bridges is given in Table 1.

TABLE 1
Deflection to Span Ratio for Mean Wind Load

Bridge Type	Main Span (m)	Width (m)	Lateral Deflection to Main-spa n Ratio (Δ/L) Without Lateral Stiffening	With Horizontal Stiffening Cable Linear	Nonlinear
Type A	50	2.5	1/323	1/3587	1/3509
	100	3.0	1/146	1/2523	1/2260
	200	7.5	1/170	1/1053	1/985
Type B	55 (100)	2.5	1/1366	1/4136	1/3892
	110 (200)	3.0	1/410	1/2904	1/2208
	220 (400)	7.5	1/477	1/1400	1/911

Note: Figures n( ) indicate the total span.

It can be seen from Table 1 that the Type- A bridges are far more wind susceptible than the Type-B bridges. However, with the stiffening cables the lateral deflections get greatly reduced. The deflection response of both type of bridges can be effectively controlled by use of the horizontal cables.

(c) Effectiveness of the lateral stiffening systems

In all, three cable truss systems and three horizontal cable systems have been investigated for providing the necessary lateral stiffness to the bridge, as shown in Fig. 2. In order to compare the various stiffening systems used, analysis has been carried out on the 100m span Type-B bridge (main span 55m) for the mean wind load. The results for mid-span lateral deflection to span ratios are given in Table 2.

TABLE 2.
Lateral Deflection to Main-span (55.0m) Ratios for Type-B 100m Span Bridge With Various Horizontal Stiffening Arrangments.

Form	Cable Trusses Linear	Non Linear	Horizental stiffening Cables Linear	Nonlinear
a	1/179 (without offsets)	1/179	1/186	1/186
b	1/183	1/183	1/222	1/222
c	1/186	1/186	1/215 (without offsets)	1/214

Note: Forms a,b & c are as shown in Fig. 2.

It can be seen from the results summarised in Table 2 that the horizontal deflection of CSP bridges can be more effectively controlled by the use of horizontal stiffening cables. Also the effectiveness of the "offsets" provided at the base of the towers to hold these cables is found to depend upon the cable arrangement. It is also seen from Table 2 that the stiffening cable systems are more effective compared to the cable trusses. The most effective system has been considered to be the one that yields the minimum value of the deflection to span ratio.

(d) Dynamic analysis :-

(i) Modal analysis

When the pipeline loads including the mass of fluids it carries, are considered, the lowest mode of vibration is the vertical mode. However, when only the self weight of the bridge without the pipeline is considered, the first mode of vibration shifts to the horizontal plane.

(ii) Earthquake response

Under the design earthquake loading, the dynamic deflection response of Type-A and Type-B cable-stayed pipeline bridges are given in Table-3. The responses have been obtained by taking S.R.S.S. of the responses in the significant modes of vibration.

The contribution of the second and higher modes were found to be generally less than 2 % .

TABLE 3.
Deflection of Cable Stayed Pipeline Bridges for Design Earthquake Loads

Bridge Type	Main Span	Without Horizontal Stiffening		With Horizontal Stiffening	
	(m)	Lateral (mm)	Vertical (mm)	Lateral (mm)	Vertical (mm)
Type A	50	28.70	17.70	10.60	25.20
	100	14.60	43.90	30.00	43.80
	200	40.20	61.20	29.30	56.90
Type B	55 (100)	20.70	22.20	10.60	22.10
	110 (200)	50.40	37.20	37.60	38.60
	220 (400)	50.20	82.50	36.90	70.40

Note: Figures in ( ) indicate the total span

(iii) Buffeting wind response

The buffeting wind response of the Type-A bridges for spans 50m, 100m, 200m for basic wind speed 44m/sec is found to be 112.5mm, 734.0mm and 178.0mm without the stiffening cables. With stiffening cables the buffeting response values are found to be 27.0mm, 148.9mm and 75.9mm respectively.For Type-B bridges of main spans 55m 110m and 220m the buffeting response is found as 46.9mm, 323mm, and 150.4mm respectively without the stiffening cables and 35.4mm, 233.0mm and 101.5mm respectively with the stiffening cables.

6. Conclusions:

Following are the main conclusions of the study.

(i) It is found that the stiffening cables are more effective than the cable trusses to control the lateral deflections of the deck.

(ii) The effectiveness of offsets used in Type-B bridges to reduced the lateral deflections is found to depend upon the cable configuration.

(iii) Unlike the traffic bridges the geometric non-linearity due to cable sag has been found negligible for the vertical load being less than 1 % .

(iv) The dynamic response for design earthquake loads is found to be significant for the cable stayed pipeline bridges

(v) Although the effective pressures across the entire span due to gusts are in themselves small, they excite large amplitude vibrations and, thereby, induce large inertia loads, which may have an effect as great as or greater than that of the mean wind.

(vi) The susceptibility of cable stayed bridges for pipelines to gusts is due to their flexibility compared to the highway traffic bridges and to the fact that their resonant frequencies concide with broad peak of the horizontal gust spectra.

(vii) Type-A bridges are found to be more buffeting prone than the Type-B bridges.

(viii) The lateral deflection of cable stayed pipeline bridges can be controlled by the use of lateral stiffening arrangements.

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