Response to Seismic Effect on Cable Stayed Bridges with different cable system under consideration of SSI

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Abstract. The prime aim of the study is to present response to Seismic effect on Cable stayed Bridges with different cable system taking under consideration SSI.It is very much known that Soil Foundation Structure Interaction relies greatly on various factors such as soil and its properties, manner and type of structure and/or its foundation. The factors like the E.Q. induced motions/vibrations are also to be considered as important too. In this paper , the emphasis is on the simplified model and foundation on piles. For the modelling authorhas used SAP2000 software. The study includes the response of the bridge modeled towards variation in the cable system under consideration of SSI. Quincy BayView Bridge is taken as a reference and 6 models are created with variation in cable system (ranging from original cable stayed bridge to suspension type, composite bridge and cable stayed suspension hybrid bridge). Soil modeling is done using the spring and dashpots (Kelvin element) for simulation of SSI effects.The results observed that effects of SSI has a substantial impact on selection of cable system for any cable stayed bridges

Keywords: Cable Stayed Bridge, Modal Time History Analysis (MTHA), Soil Structure Interaction (SSI) , SAP2000.

1 Introduction

In the current socio-economic criteria Bridges are among the top in the list critical lifeline services. And as such need of long span bridge has accumulated with boom of infrastructure. the want of unbelievable bridges of lengthy span is amassed after every passing day due to increase in population inhabiting across the globe, leading to increase importance for usage of material(s) with high to ultra high strength blended with innovative structural system .

A few captivating and breathtaking bridges have been structured and worked over the most recent couple of decades than at some other similar time throughout the entire existence of development. The systems of cable supported bridges for the most part used to accomplish longer lengths can be easily categorized as Cable Stayed

Bridges (CSB's), Suspension Bridges(SB's), Composite Bridges (CB's), Cable stayed Suspension Hybrid Bridges (CSSHB's)

In normal, to garner longer span bridges, CSB's and SB's are preferably selected/provided.CSSHB possesses superiority in preference SB's and/ or CSB's due to the fact it comprises advantages of each cable stayed nonetheless as suspension bridges.

Bridge, being long to super long structures for communication , its failures may/can lead to great and higher level of damage and loss to life and material comparable to that of catastrophic failures. This booms in the concept of prevention of such catastrophic accidents which may be possible by properly understanding the reasons that result(s) failure of bridges.

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Reasons for failures of Bridges

The top and foremost reasons because of bridges become seriously compromised or collapse can be categorically listed as ::

- Failures occurring during construction
- Failure occurring while bridge in service (in absence of any external action)
- Collapse in event of impact(may be due to collision)
- Failure caused due to onset of cyclone, tsunami, hurricane, flooding, ice or other-floating objects; fire or explosion; <u>seismic activity;</u> falsework; incompetent design
- Combination of more than one of above

As a summary, causes commonly attributing to failures of bridge can be classified in a broader sense as due to basic and plan inadequacies, erosion, development and supervision botches, unintentional over-burden and effect, scour, and absence of upkeep or review

Prevention is the best cure concept leads to the methodology reflecting best way to avoid bridge failures. This attitude incorporates the concept of expecting them(failures) to happen and plan for them. It proves to be one of the wayto protect the public from injuries, loss of life, property damage, and destruction. This enhances the Interest in learning about ways to improve and update /upgrade bridge design and quality of construction and constructional practices adopted.

1.1 Soil Structure Interaction (SSI)

Thus, Seismic Soil Structure interaction (SSI, hereafter) plays an important element in the understanding of seismic structure failure.The damage caused with foundation of bridges in earthquake(s) has emphasized the importance of understanding SSIcompared to free-field motions.Soil–structure interaction (SSI) thus is an important issue and must not be ignored in the seismicdesign of important structures including bridges. Thus, SSI, a complicated phenomenon [1], involving Wave Amplification ::::Seismic waves alter due toflexibility of soil Kinematic Response ::::Soil displacement is causingstructure to displace Inertial Reactions:::::::: Stress movement continuesand adds to displacement of structure

This procedure (named SSI), in which the reaction of the soil impacts the movement of the structure and the movement of the structure impacts the reaction of the soil presumes that both, soil and structure, are associated and not independent from one another

SSI can be sub categorized[2] as Static SSIKinematic or Dynamic SSIAlso, Soil-structure interaction[3],[4]can be broadly divided into two phenomena: Kinematic interaction Inertial interaction

Effects Soil Structure Interaction (SSI)

SSI changes the dynamic qualities of the structural reaction essentially. These impacts were disregarded in the past however because of the failure of such huge numbers of enormous structure during seismic tremor occasion the essentialness of SSI was figured it out. In this manner SSI is given significance and bunches of research work is proceeding to think about the impacts of SSI on different structures is condensed as

Alter the Natural Frequency of the StructureAdd Damping

Through the Soil Interaction effects

Travelling Wave effects

2 Literture Survey

Examinations were completed in different investigations with respect to the impact of SSI on the tremor reaction of a few ordinarily structured extensions lately

Spyrakos (1990, 1992)demonstrated that SSI enormously influences the seismic reaction of bridges driving toward progressively adaptable frameworks and expanded damping by using basic models which are linearly flexible.

Ciampoli and Pinto (1995) explored on parametric examination on traditionally structured bridges established on shallow establishments thinking about inelastic reaction of the piers. Information of Eurocode good falsely produced accelerograms (far field excitations) were considered and reasoned that SSI impacts reliably diminished the pliability requests of the piers when contrasted with the system without SSI impacts

Jeremic et al, (2004) contemplated in detail the seismic reaction of the I-880 viaduct in Oakland, Calif. To conclude that “SSI can have both beneficial and detrimental effects on the response of the structure depending on the characteristics of the ground motion”

Zhang (2004) conductes investigation on the effect of SSI on the response of 9/15 Overcrossing in Los Angles and abstracted that ignoring SSI would lead to an underestimation of seismic forces

Tongaonkar and Jangid (2003) assessed the effects of SSI on three-span continuous deck bridges isolated with elastomeric bearings. They performed MTHA , assuming linear elastic behavior for the isolation system and the piers of the bridge to concluded that consideration of SSI in the analysis will result in the enhancement of safety and reduction in design costs. They supllimented that under certain circumstances, isolation bearing displacements at abutment locations only might be underestimated if SSI is not accounted for in the analysis.

Mylonakis and Gazetas (2000)took in thought a lot of real acceleration time histories recorded on delicate soil, utilizingan improved model for the bridge and its foundation, and presumed that the period protracting and expanded damping due to SSI impacts can detrimentally affect the forced seismic demands

Examination Between Three Types of Cable Stayed Bridges Using Structural Optimization. In his work, a progressed and extensive numerical model was utilized to get the post-tensioning forces and the ideal structure of the three sorts of link/cable stayed bridge. The numerical technique dependent on limited component, B-spline bends, and genuine coded hereditary calculation was embraced. The improvement represents every one of the factors that characterize the geometry and cross-segment of the bridge. Correlation between the three sorts, as far as post-tensioning forces and cost, was completed

Siddharth Shah et al (2011) given investigation in which spotlight is given on the impact of pylon's shape on the seismic reaction of cable stayed bridge. The examination uncovers that the pylon's shape has extraordinary impact in the seismic reaction of cable stayed bridge. spread shapeof the pylon are better for opposing seismic tremor longitudinal direction however feeble sidelong way, yet pyramid shapeof the pylon is better a result of its geometry in opposing quake power from any direction and furthermore SSI impacts are least for this situation. SSI impacts are transcendent for delicate soil conditions for all shapes of the pylon

Tao Zhang(2011) summarized that forces in the cables are significant in design of cable stayed bridges . By investigating a basic auxiliary system, the methodology utilizing the examination program MiDAS was delineated. The model was generated for the completed dead stage examination was outlined in detail, including the boundary and load combinations. The streamlining technique for onknown load factor was utilized to decide the forces in the cables to accomplish a perfect state. The perfect cable force is built up and a development stage analysis is performed. The greatest cable forces were demonstrated to be in limits permissibility. The outcomes got uncovered that the technique exhibited forsure prompts ideal performance of the structure for the cablestayed bridge specifically, and may be a valuable reference for the plan of other comparable bridges

John C. Wilson,WayneGravell(1991*)* studied Modelling of a cable- stayed bridge for dynamic analysis and in his study, provides a detailed description of the development of one class of linear elastic finite element model for the dynamic analysis of a cable- stayed bridge. The bridge modelled in this study is the Quincy Bayview Bridge in Illinois..

2.1 Introduction to the software

In the present study, thesoftware used is SAP2000 v 20.2.1. It is a product of CSI,Berkely,USA.It is used for analyzing general structures ranging from bridges to stadiums, dams to industrial buildings, offshore to onshore structures, soil etc. It has fully integrated programme that allows model creation, editing(modification),execution of the analysis, design optimization, review of results etc from within a single interface

SAP 2000 is a comprehensive and integrated design and finite element analysis Tool.It offers features like

- Multiple Coordinate system ;Powerful graphical display
- Frame, Cable & Shell structural elements
- Wide range and variety of loading options including loading functions of Time History, Response Spectrum etc.
- Static & Dynamic Analysis; Linear & Non-linear Analysis; Dynamic Seismic Analysis & Pushover Analysis ; Geometric Nonlinearity including P-δ effect ; Nonlinear Link & support analysis
- Frequency dependent link & support properties

3 Problem Studied

B.1 Quincy Bay View Bridge (Type I CB).

In the present study a cable stayed bridge considered is similar to Quincy Bay View Bridge

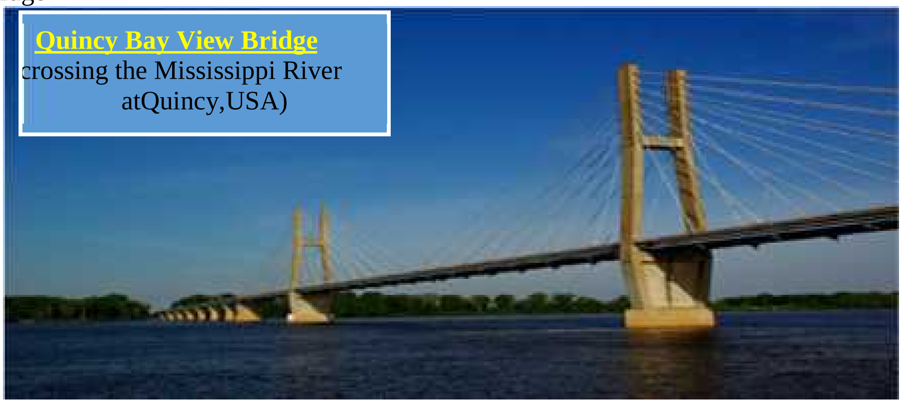


Fig. 1. The Quincy Bayview Bridge

The Quincy Bayview Bridge, shown in **Figure 1,** above was designed in 1983, and construction was completed in1987. The bridge consists of two H-shaped concrete towers, double-plane fan type cables, and a compositeconcrete-steel girder bridge deck. The main span is 900 ft (274 m) and there are two equal side spans of 440 ft(134 m) for a total length of 1780 ft (542 m). The tops of the towers are 232 ft (71

m) from the waterline. There are a total of *56* cables, 28 supporting the main span and 14 supporting each side span. The width of the deck from centre to centre of cables is 40 ft (12 m).

A typical cross-section of the actual bridge deck is shown in **Figure 2.** It is a simplified deck cross-section between anchor points that was used to evaluate physical properties for the model. The road deck is a 9 in' precast post-tensioned concrete slab 46.5 ft wide with two non-structural precast parapets (traffic barriers). Five longitudinal steel stringers are spaced at equal transverse intervals of 7.25 ft. Floor beams transverse to the main girders at equally spaced intervals of 30 ft transfer stringer loads to two main girders at the outer edges of the deck. The cables are connected to the deck at the bottom flange of the main girders.

Each tower, consists of two concrete legs, with dimensions of 14.5 x 7 ft (the largerdimension is in the longitudinal direction of the bridge), a lower strut (cross-beam) supporting the deck andan upper strut connecting the upper legs. There are three changes in the leg cross-section over the height ofthe towers.

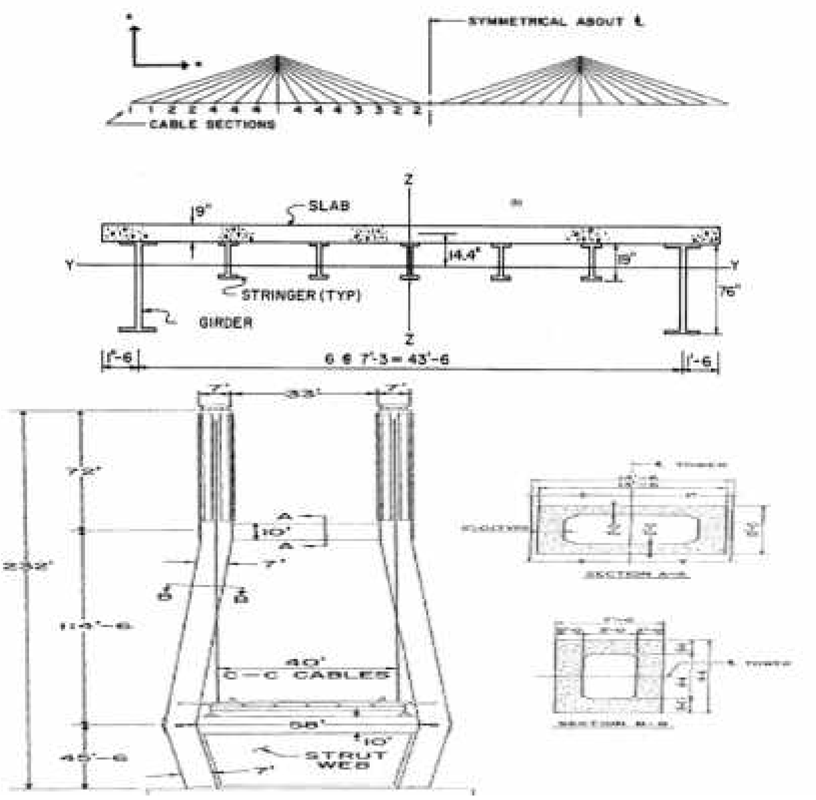


Figure 2..Elevation, c/s& cable system of QuincyBayView(Type I)[5]

To avoid expansion joint the bridge is modeled with roller joint supports at cantilever end (side spans) whereas the deck pylon support is roller and hinged support respectively along the longitudinal direction

B.2 Quincy Bay View Bridge (Type II CSB)

This is a bridge with modified cable system, hence called Composite Bridge (Type II CSB) hereafter

In this the central span is converted to SB whereas the side span s are cable stayed as shown in Fig. 3(b) hereafter. The material for hangers as well as suspension cable is same as that of stays. However the diameter of suspension cable is taken as 0.4m @ 9.613 kN/m whereas hangers are of 0.1069m diameter @ 0.263 kN/m respectively. Sag to central span ratio is taken as 1/6.The hangers are placed/connected to deck at same respective location(s) where cable stays were connected to the deck in the originally designed CSB.The side spans are same as original CSB with 14 stay cables on each side as depicted in **Figure 2** and **Figure 3 (b)** respectively

B3 Quincy Bay View Bridge (Type III CSB)

This is again a bridge with a modified cable system, hence called Composite Bridge (Type III CSB) hereafter.In this the central span is converted to CB whereas the side span s are cable stayed as shown in **Figure 3(c)** .The material for hangers as well as suspension cable is same as that of stays. However the diameter of suspension cable is taken as 0.4m @ 9.614 kN/m whereas hangers are of 0.1069m diameter @ 0.631 kN/m respectively. The hangers are placed/connected to deck at same respective location(s) where cable stays were connected to the deck in the originally designed CSB(side spans).There are a total of *28* cables in the central span with hangers spaced at 19m c/c in the side span(suspension type)

B4 Quincy Bay View Bridge (Type IV CSB)

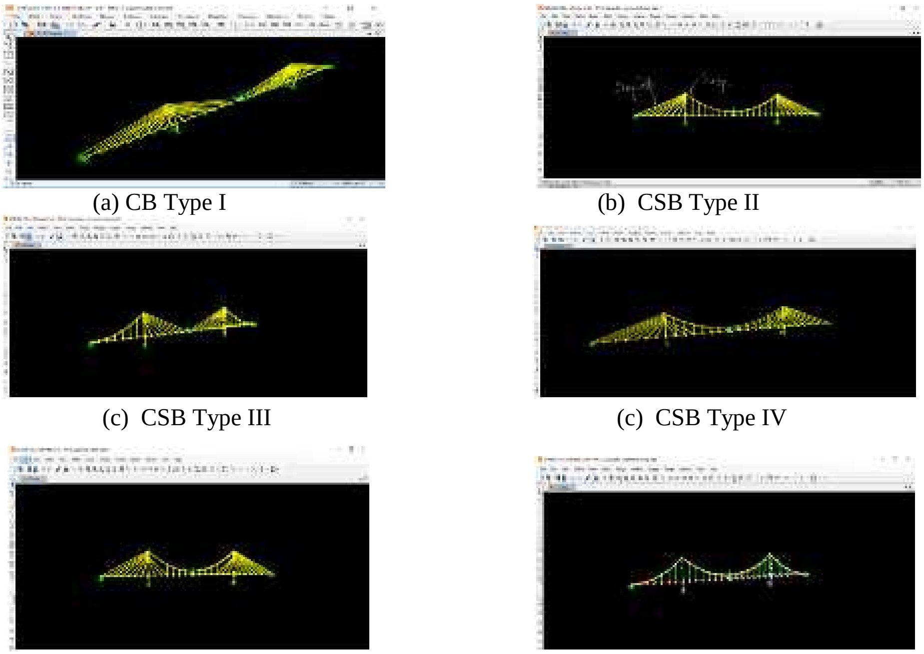
In this the central span is converted to CB whereas the side span s are cable stayed as shown in **Figure 3(d)** hereafter.The material for hangers as well as suspension cable is same as that of stays. However the diameter of suspension cable is taken as 0.4m @ 9.614 kN/m whereas hangers are of 0.1069m diameter @ 0.631kN/m respectively. Sag to central span ratio is taken as 1/6.There are a total of 28 cables in the side spans and 12 in the central suspension portion with hangers spaced at 19m c/c in the central spanthroughout.In the central span , the deck is further stiffened by providing 3 cable stays on each side,i.e total 12 cable stays as shown in **Figure 3(d)**

B5 Quincy Bay View Bridge (CSSHB type)

In this the central span is converted to CB whereas the side spans are cable stayed as shown in **Figure 3(e)** hereafter.The material for hangers as well as suspension cable is same as that of stays. However the diameter of suspension cable is taken as 0.4m @ 9.614kN/m whereas hangers are of 0.1069m diameter @ 0.631 kN/m respectively. Sag to central span ratio is taken as 1/6. The hangers are provided at same points where there were stay cables in CB type i.e hangers are placed @ 19m c/c

B6 Quincy Bay View Bridge (Suspension Bridge –SB type)

In this the bridge is modlled by converting it to SB as shown in **Figure 3(f)** hereafter. The material for hangers as well as suspension cable is same as that of stays. However the diameter of suspension cable is taken as 0.4m @ 9.614kN/m whereashangers are of 0.1069m diameter @ 0.631 kN/m respectively. Sag to central span ratio is taken as 1/6.There in hangers are placed at the same points where cable stays were attached to girder in case of original Quincy BayView CB (i.e with hangers spaced at 19m c/c)



(a) CB Type I

(b) CSB Type II

(c) CSB Type III

(c) CSB Type IV

(e) CSSHB Type

(f) SB Type

Figure 3. : Bridges with different cable systems

C1 Modelling of Bridge Structure

.

CSB like any other structure is divided in 2 main components namely Super-structure and Sub-Structure. For Finite Element Modelling of CSB, propertiesof material used and sections considered are entabulated in subsequent tables below.he finite element is developed using properties of material and section entabulated in **Table 1** below

Table 1.:Material used ,Cables& sections considered

Property	Steel	Concrete
Modulus of Elasticity (*E*)	2.1×10^{8} kN/m^2	2.985×10^{7} kN/m^2
Unit Weight	76.973kN/m^3	24.993kN/m^3
Poisson's ratio (μ)	0.3	0.25
Shear Modulus (G)	8.077 x10^{7} kN/m^2	1.232 x 10^{7} kN/m^2
Coeff. Of Thermal Expansion (α)	1.17 x 10^{-5}	0.55 x 10^{-5}

a) Material used

Cable No.	Diameter (m)	Area(m^2)	Cable weight(kN/m)
1	0.1069	8.918×10^{-3}	0.686
2	0.0946	6.984×10^{-3}	0.537
3	0.0827	5.337×10^{-3}	0.411
4	0.0666	3.416×10^{-3}	0.263

b) Cables used

Component	Dimension	Material	Shape
Deck..end beams	h = 1.90m ,bf = 0.62m tf = 0.08m,tw = 0.04m	Steel	I- section
Deck ..Stringer beams	h = 0.50m,bf = 0.32m tf = 0.02m,tw = 0.01m	Steel	I- section
Stiffening wall	1.2m thick	Concrete	Rectangular
Pylon bottom	(2.1336 x 4.4196)	Concrete	Solid Rectangular
Pylon intermediate	(2.1336 x 4.4196)- 0.9144 m dia hole	Concrete	Hollow Rectangular
Pylon intermediate	(2.1336 x 4.4196)- (0.7897 x 0.9144) m	Concrete	Hollow Rectangular Box

c) Sections considered

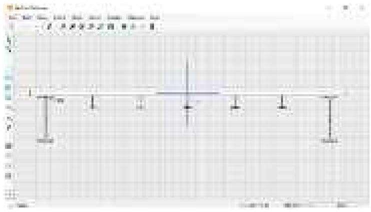


Figure. 4. ::C/s - deck girder

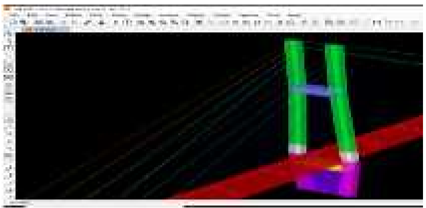
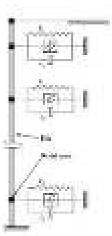
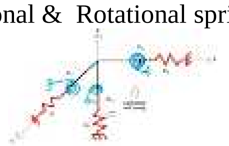


Figure 3 .:: 3-D extruded view of Pylon, Trans verse beams &Stiffening walls

C2 Modelling of Soil

The interaction between the pier footing and the soil is modelled using translational & Rotational springs**.**





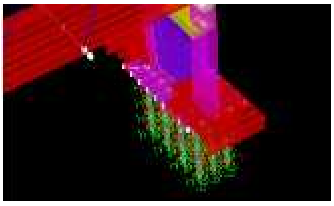


Figure 6 :: Modeling of soil as spring & Dashpot-(Kelvin Element) applied at nodes of pile[7] (Adopted from Soneji, B. B & Jangid 2009)

Figure 7:: Soil 1 (Hard Clayey) modelled

The spring coefficients have been computed by the method suggested in Specification for Highway Bridges issued by Japan Road Association. In the suggested method, it should be mentioned that, when using equations (1) and (2), the units of B_e and E must be centimeters and kgf/cm^2 (1 kgf/cm 2 = 98 kPa), respectively. The horizontal and rotational spring coefficients for each part of foundation are obtained by multiplying k by the area and the inertia moment of its surface perpendicular to the excitation direction, respectively. As for the bottom face of foundation, the soil reaction coefficient per unit area in horizontal direction is taken as 1/3 of k.

k_0 = 1.2E /30……………………………………………..(1)

k= $k_0^{-0.75}\sqrt{}(B_e/30)$…...……………………………………(2)

Where,

k_0= reference soil reaction coefficient,

E=Young's modulus of elasticity for soil,

k =The soil reaction coefficient per unit area,

Be= the width of foundation perpendicular to the considered direction.

Three types of soils are in this study designated as soil type I,II,III in the table below.

Table 2::Different Properties including lateral & rocking stiffness coefficients [1]

Sr No	Soil Properties	Soil I::Hard Clayey soil	Soil II::Soft Silty Soil	Soil III::Med. Sandy soil
1	Unit Wt.,ϒ ,kN/m^3	20	18	19
2	Shear Strength,τ_s , kN/m²	200	75	150
3	Poisson's ratio,	0.3	0.4	0.35
4	Dampingof soil, ξ	0.02	0.06	0.04
5	Shear Wave velocity ,Vs (m/s)	1050	83	309
6	Shear Modulus ,G,kN/m²	269 x10^4	12500	192310
7	Young's modulus ,E,kN/m²	700 x10^4	35000	500 x 10^3
8a)	Soil Stiffness, Kx(kN/m)	252 x10^4	4.60x10^4	8.62x10^4
8b)	Soil Stiffness, Ky(kN/m)	1050 x10^4	5.52x10^4	10.3 x10^4
8c)	Soil Stiffness, Kz(kN/m)	1028 x10^4	5.36x10^4	10.0 x10^4
8d)	Soil Stiffness, Kθx(kN/m/rad)	8094 x10^4	156 x 10^4	292 x10^4
8e)	Soil Stiffness, Kθy(kN/m/rad)	309 x10^4	21.6 x10^4	40.4 x10^4
8f)	Soil Stiffness, Kθz(kN/m/rad)	9808 x10^4	532 x10^6	729 x10^6

C3 Details of acceleration Time History

• Name : Bhuj
• Magnitude : 7.7
• Duration : 133.53 seconds
• Peak Ground Acceln. : 1.0382 m/s²
• Total No. of Acceln.records : 26706

4 Analysis and results

For evaluation of the seismic response , using Bhuj Earthquake near fault data, Time History Analysis (THA) was performed first on Quincy BayView Cable stayed bridge(CB_Type 1). This was followed subsequently on various bridges modeled as shown in Figure 3.All the models are analysed subsequently to study the impact with and without SSI. For ascertaining SSI, 3 types of soils are considered as mentioned previously.The piles upto depth of 20m is considered for all models (for SSI).Result(s) Table below , clearly reflects that bridge modelled (Quincy Bay) in the Sap 2000 give the results which are found similar with results presented in the literature (research paper referred)..

Table 2::Time Period(1^{st} mode—CB Type I) different researchers

Modes	Paper Results (Wilson &Gravelle)		Our Results (SAP2000)		% Error	
	T (sec)	f (Hz)	T (sec)	f (Hz)	T (sec)	f (Hz)
Mode -1 (Lateral)	2.695	0.371	2.821	0.354	4.67	-4.58

The results of seismic time history analysis , as entabulated in **Table 3** for 24 cases (bridge and soil type). The table demonstrates the change in time period with change in stiffness of soil underneath.

Table 3. Modal Time Periods : Bridges

1^{ST} MODE TIME PD, SEC., FOR DIFFERENT CASES (SOILS) (FOR 20M DEPTH OF PILES (FOUNDATION))							
Bridge Type	W/O SSI	With SSI					
	(Fixed Base)	Soil I	% change	Soil II	% change	Soil III	% change
CB TYPE I	2.8214	2.3981	-15	2.4107	-14.56	2.4002	-14.93
SB	3.9046	4.6735	19.69	4.6821	19.91	4.6774	19.79
CSB TYPE II	3.89	4.6495	19.53	4.6512	19.57	4.6498	19.53
CSB TYPE III	2.8291	2.4735	-12.57	2.5701	-9.16	2.4784	-12.4
CSB TYPE IV	3.028	3.6271	19.79	3.6328	19.97	3.6299	19.88
CSSHB TYPE	3.1958	3.8265	19.74	3.8312	19.88	3.8287	19.81

5 Conclusion

The results of seismic time history analysis , as entabulated in TABLE 3 for 24 cases (bridge and soil type). The table demonstrates the change in time period with change in stiffness of soil underneath. It depicts that the trend of change in time period (1st mode) follows almost the same trend wrt to bridge type for various cases. The increase in max in case of Suspension Bridge type (SB) and composite bridge type II (CSB) and has a decressed time period in case of CSSHB type .Thus SB and CSB type II have proved to be more flexible (has decreased stiffness) when compared to cable stayed bridge.Composite bridge Type IV has more or less same stiffness, while csshb bears stiffness between CB and SB.

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