

Behavior Study of Two Decked Cables Stayed Concrete Bridge Under Modal Frequency Analysis

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Abstract:

Two decked cable stayed concrete bridges are engineered to support heavy traffic loads while optimizing space utilization in regions with limited land availability. These bridges are particularly suitable for spanning large distances while maintaining high functionality and aesthetic appeal. This research investigates the dynamic behavior of such bridges by utilizing finite element analysis (FEA), with a focus on modal frequencies, oscillation periods, and mode shapes that characterize the vibrational response pertaining to the structure. The Comprehensive Two decked cable stayed concrete bridge was devised by using staad pro to simulate the interactions between key structural components, including 12 piers, 1440 plates, 2 H type pylons, 160 cables. The modal mass participation analysis revealed the influence of elementary vibrational modes on the overall dynamic behavior of the structure, emphasizing the importance of mass distribution and dynamic loading effects in the performance of the bridge. Parametric studies were conducted to assess how changes in damping ratios affect the inherent frequencies of the structure. The time history distribution was performed to evaluate the bridge's behavior under transient loading conditions. The analysis offered valuable insights into how damping helps control vibration amplitudes at Node 2 during the 1, 2, 3, 4, 5, 6 second time interval. Through interpretation of the graphs of acceleration, velocity, and displacement over time, it became apparent that damping plays a pivotal role in reducing vibrations—ultimately helping the bridge better withstand fatigue from repeated loading. This study yields noteworthy insights that help improve bridge design strategies, durability of double-deck cable-stayed concrete bridges, and support their long-term stability and resilience—especially when facing dynamic forces and the effects of maximum base shear in X, Y, Z directions."

Keywords: Two decked cable stayed concrete bridge, Time history distribution, modal frequency, Staad pro, Material Formulations.

1. INTRODUCTION:

Expedition of bridge technology is interrelated with evolution of community, spanning thousands of durations. One of timeworn surviving illustrations pedestrian stone bridge over Meles River in modern-day Turkey believed over 2,800 years old. Records of the past from writers like Homer and Herodotus mention floating spans made from inflated animal hides, used as early as 800 B.C. Around 320 B.C., Alexander the Great utilized comparable buoyant constructions to aid his troops in traversing rivers during each eastern expedition. In 200 B.C., Romans conquered use of stone arches supported by robust piers—a highly notable leap in structural engineering. A standout example from Renaissance: Venice's Rialto Bridge, built 1591, parading graceful single-span design measuring 27 meters.

The advent of contemporary bridge engineering commenced 1714 when French Henri Gautier engineer Published first comprehensive text on subject. Momentum carried on with founding of École des Ponts et Chaussées in Paris, 1747—world's first engineering school. Rodolphe Perronet, considered father of modern bridge construction, refined masonry arch design. Pont de la Concorde in Paris (1791) remains one of his most recognized works. Firth of Forth Cantilever Bridge (completed 1890) and John Roebling's Brooklyn Bridge, with 490-meter main span, exemplify this advancement. Post-World War II, reinforced concrete gained popularity

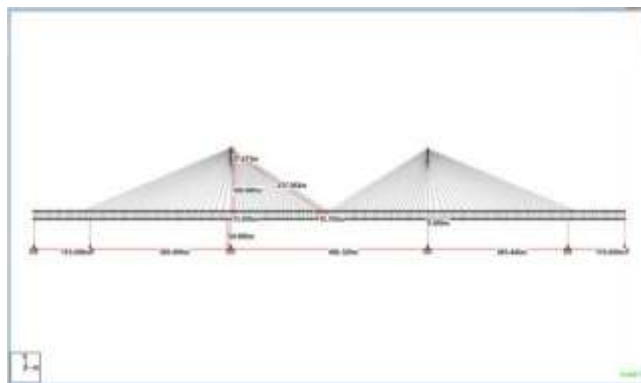
for durability, low maintenance—especially in short to medium spans. In 1928, French technology Eugène Freyssinet unveiled prestressed concrete, enabling longer, more economical spans. By 1950s, around 70% of new bridges in Germany used this technique. A modern example Ganga Bridge in Patna, India, utilizing precast segmental box girders, standing as Asia's longest prestressed concrete bridge. Another milestone: cable-stayed bridge, where inclined cables support deck. First modern example: Strömsund Bridge in Sweden completed 1955. This study proposes a new shear connector gusset connection for seismic design of corrugated concrete frames with buckling-restrained braces, demonstrating bettered seismic performance and damage control[1]. This study examines the dynamic geste of double- sundeck string- stayed islands under train loads on the lower sundeck, assaying factors like weight combinations, train faves, and damping rates to give perceptivity for ground design and conservation[2]. former studies on brand - concrete conflation joints(SCCJs) have primarily concentrated on their structural integrity under static and dynamic loads, but limited attention has been given to their performance in high- speed road crossbred string- stayed islands. Recent exploration has emphasized the significance of stress distribution, weight transfer mechanisms, and distortion geste in icing structural safety and continuity. This study builds on that foundation by combining experimental and numerical styles to estimate SCCJ performance under extreme weight conditions [3]. Former exploration on innovative brand - concrete joints (SCJs) in long- span road islands has emphasized the significance of stress distribution, slip geste , and axial force transfer mechanisms in maintaining structural integrity under high loads. Finite element analysis (FEA) has proven essential in validating experimental findings and optimizing common design, particularly regarding bearing plate effectiveness and weld detailing. This perceptivity directly inform the dynamic modeling and modal frequency analysis of complex systems like double- sundeck string- stayed concrete islands using STAAD.Pro [4]. Former exploration on orthotropic brand sundeck (OSD) islands has explored structural performance under colorful functional countries, but multitudinous studies integrate lifecycle assessment (LCA) with structural optimization. Recent work emphasizes how material choice, opening angles, and actuation systems impact both mechanical demands and environmental sustainability. This study advances that understanding by linking high-grade brand use with mass reduction, bettered performance, and reduced CO₂ emigrations in bascule ground design[5]. Former studies on ground sundeck fatigue have primarily concentrated on orthotropic brand sundecks (OSDs), with limited attention to the part of ultra-high- performance concrete (UHPC) in enhancing fatigue resistance. Recent exploration highlights UHPC's eventuality to significantly reduce stress ranges and diversions in conflation sundecks, thereby extending service life. This study contributes by validating field-covered fatigue data and finite element analysis, emphasizing UHPC's effectiveness in real business conditions [6]. Several studies have explored the dynamic geste of string- stayed islands, emphasizing the influence of sundeck wind, string arrangement, and palace indolence. Zhang and Xu(2005) detailed modal identification under functional conditions, while Frýba(1996) mooted vibration modes and their perceptivity to geometric configurations. Recent advancements, similar as vision- rested monitoring (Farrar & Worden, 2012), farther support accurate remote dynamic assessment [7]. Recent exploration highlights the eventuality of fiber corroborated polymer(FRP) lines in long- span string- stayed islands, offering superior erosion resistance and reduced weight(Nakamura et al., 2000). Studies by Kim and Meier (1991) demonstrate the structural and profitable benefits of crossbred FRP arrangements. This paper extends those findings by proposing a span- optimized crossbred FRP system that enhances seismic performance and reduces life-cycle costs [8]. Cable- stayed footbridges are largely susceptible to dynamic issues like coterminous side excitation, as mooted by Dallard et al. (2001) following the Millennium Bridge case. Studies by Caetano et al.(2010) emphasize the significance of integrating structural and damping system design. This paper builds on those principles by proposing an optimization algorithm that contemporaneously addresses structural, dynamic, and profitable performance [9]. Crooked string- stayed islands(CSBs) are decreasingly espoused for aesthetic and point-specific conditions, but their seismic design remains less explored(Kim & Laman, 2010). Formerly exploration, similar as by Zhang et al.(2015), highlights the perceptivity of CSBs to seismic goods, particularly spatial ground stir variability. This study advances the field by integrating construction stages and seismic optimization using an entropy- rested algorithm [10]. Recent studies emphasize Ultra-High Performance Concrete(UHPC) overlays as a promising result to fatigue issues in orthotropic brand sundecks(OSDs), enhancing stiffness and reducing stress attention(Graybeal, 2006). exploration by Liu et al.(2020) highlights UHPC's effectiveness in extending fatigue life at critical welded joints. This study advances understanding by combining in- situ measures and FE modeling to estimate residual fatigue life in brand- UHPC conflation sundecks [11]. Construction simulation of string- stayed

islands is critical due to high stress situations during construction, constantly exceeding those in service (Virlogeux, 1999). Traditional styles calculate on stage superposition and iterative processes, limiting harshness and adding computational demand (Kawashima & Unjoh, 1997). This study introduces a direct analysis(DA) approach using the unstressed length system, enhancing effectiveness and delicacy without specialized software[12]. Crossbred string-stayed suspense islands(HCSBs) offer enhanced gauging capability but are prone to fatigue in hanger zones due to cyclic stresses(Xu et al., 2020). former studies have explored structural parameters affecting fatigue, but detailed evaluation using influence line styles remains limited. This exploration provides new perceptivity into optimizing hanger layout, stiffness, and string arrangements to reduce stress confines and meliorate fatigue resistance [13].

2. Methodology:

This investigation entails the modeling of a two-decked cable-stayed concrete bridge using STAAD.Pro CONNECT Edition V24 to scrutinize its dynamic characteristics. A modal frequency analysis was conducted to determine the structure's natural frequencies and mode shapes. The materials and loading criteria were defined in accordance with IS 456:2000 and IRC:6-2017 standards. Support conditions and cable forces were assigned based on conventional bridge engineering methods. The analysis results offer valuable Apprehension of the bridge's vibration behavior, contributing to the Maturation of safer and more effective structural designs.

2.1Two decked cable stayed concrete bridge schematic: A two decked cable stayed concrete bridge with spatial attributes of span 1200 meters, upper deck 16 meters, lower deck 8 meters shown in **fig(1)** engaged for assessment



2.2Structural attributes

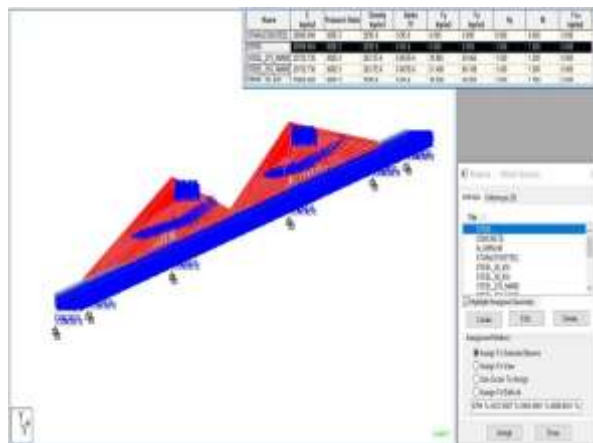


Fig:2 Structure cable's properties

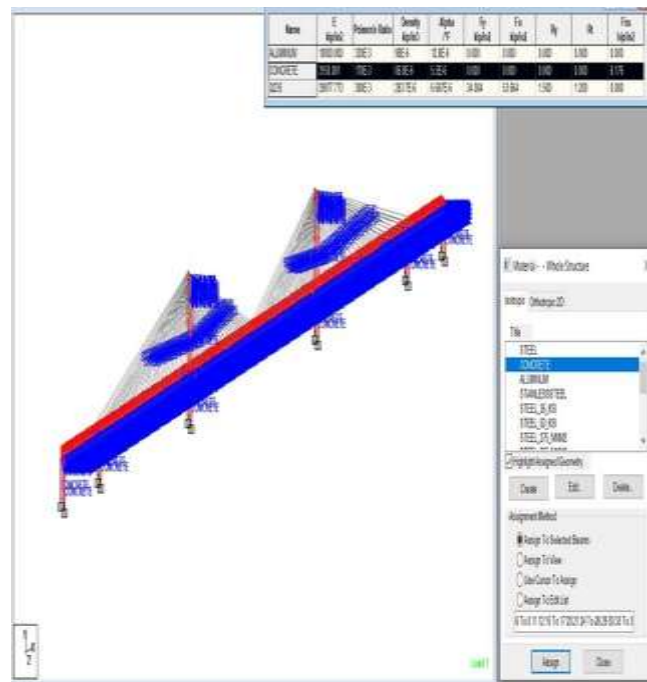


Fig: 3 Structure deck, pier and pylon properties

2.3 The structural loads are detailed below.

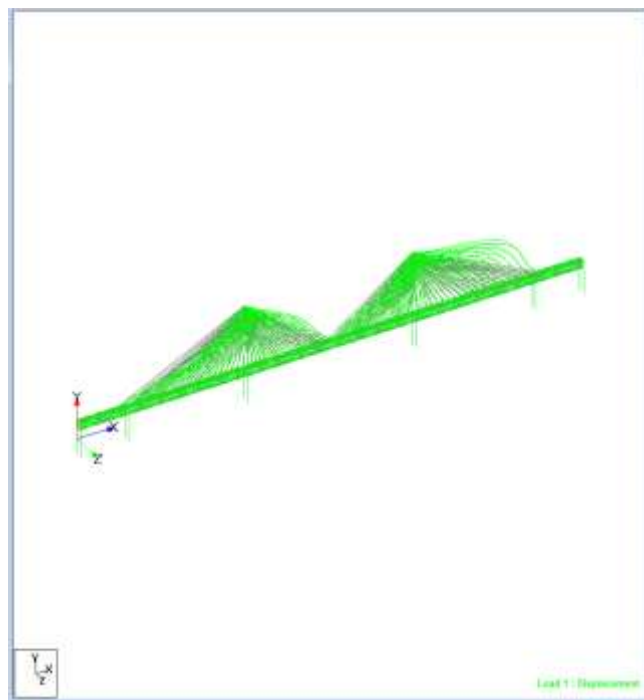


Fig: 4 shown in fig displacement of plates and beams under modal load



Fig: 5 Shows the loads applied

2.3.1 Time – displacement response of the two decked cable stayed bridge

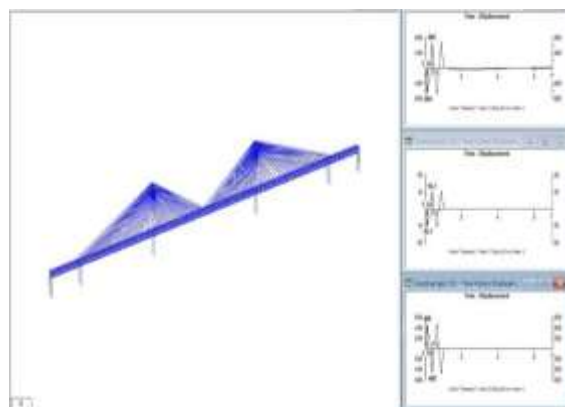


Fig: 6 shown in fig time - displacement of response in graphs at node 2

The time history analysis shows significant transient displacement at key nodes, with peak responses occurring around 0.376 seconds. Displacements rapidly decay, indicating effective damping in the structural system.

2.4 The calculated frequencies under load case 2 (5.0625 Kn/m)

Mode	Frequency(cycles/sec)	Time period in sec
1	0.055	18.14442
2	0.059	17.01067
3	0.065	15.37207
4	0.083	12.01518
5	0.093	10.72283
6	0.094	10.63868
7	0.098	10.25129
8	0.102	9.79996
9	0.102	9.75776

Table 1: The calculated frequencies under load case 2

Mode 1 shows the lowest frequency (0.055 Hz) and the longest period (18.14 sec), reflecting the slow, flexible movement typical of the primary vibration mode in a cable-stayed bridge. Modes 8 and 9 show the highest frequency (0.102 Hz) and shortest periods (~ 9.8 sec), representing faster, higher-order vibrations.

2.5 Mode Contributions to Resonant Response in the Bridge

Mode	Frequency(Hz)	Importance
1	0.05513	0.0000003931
2	0.058787	0.0000500650
3	0.065053	1
4	0.083228	0.0000500650
5	0.093259	0.0031792000
6	0.093997	0.0488756514
7	0.097549	0.0086674653
8	0.102041	0.0792858147
9	0.102483	0.0000233484

Table 2: Contribution of Missing Mass and Mode Importance at Resonance

For the two-decked cable-stayed bridge, Mode 3 plays a key role in how the structure responds to resonance. Even though its frequency is relatively low at 0.065 Hz, it carries the highest importance value of 1.000, showing that it dominates the dynamic response. On the other hand, Mode 9, despite having a higher frequency of 0.102 Hz, contributes very little, with an importance value close to zero (0.00002). Modes 1, 2, and 4 also have extremely low importance, meaning they have only a minor effect on the bridge's movement when exposed to dynamic forces at resonance

2.6 Reactions and Moments at Supports in the X, Y, and Z Directions

Postprocessing: Displacements Reactions Beam Results Plate Results Solid							
All / Summary / Envelope /							
Node	LIC	Horizontal Fx kN	Vertical Fy kN	Horizontal Fz kN	Moment		
					Mx kip-in	My kip-in	Mz kip-in
2665	6 DL	-62 439	88103 781	-76 400	-12260 151	-17366 041	23564 863
	7 LL	-29 518	29786 192	-27 427	-4399 936	-7889 893	10873 426
	1 MODAL LOA	-46545 379	371 61052E3	-74841 842	-18 53623E6	390 00884E3	14 37414E6
	2 THD	2832 222	-212 396	1 590	199 942	-14154 881	-552 61244E3
2667	6 DL	-62 470	88106 118	-76 216	12214 385	17368 395	23570 711
	7 LL	-29 512	29786 318	-27 409	4395 303	7889 437	10872 223
	1 MODAL LOA	-48659 888	-529 18547E3	-75055 331	-18 57050E6	139 46172E3	14 91879E6
	2 THD	2832 498	-219 544	-2 804	-505 086	14096 917	-552 67856E3
2678	6 DL	63 953	87978 379	-76 752	12311 533	-17497 936	-23542 656
	7 LL	30 184	29794 176	27 654	4437 034	-7944 219	-10821 580
	1 MODAL LOA	-46656 463	-534 94665E3	-74913 804	-18 53981E6	-356 95222E3	14 41707E6
	2 THD	2807 656	321 577	2 240	324 948	14250 777	-547 63475E3
2680	6 DL	63 521	87987 840	-76 654	-12289 721	17466 360	-23459 196
	7 LL	30 144	29794 997	-27 618	-4428 268	7940 890	-10813 761
	1 MODAL LOA	-48770 685	360 79787E3	-74842 572	-18 52843E6	-173 19592E3	14 95389E6
	2 THD	2808 637	318 497	-3 010	-516 295	-14135 447	-547 83962E3
2694	6 DL	-2418 494	14549 177	-381 387	-61147 617	6830 454	374 65347E3
	7 LL	-1077 023	3986 009	-162 213	-26002 021	3046 941	166 62487E3
	1 MODAL LOA	-27849 265	27455 080	-12037 457	-2 60105E6	-349 10625E3	11 62494E6
	2 THD	984 373	-2749 045	106 688	17078 662	1873 096	-259 81172E3
2696	6 DL	-2418 463	14549 218	381 094	61073 199	-6843 814	374 64344E3
	7 LL	-1077 025	3985 912	162 216	26002 953	-3044 713	166 82561E3
	1 MODAL LOA	-25228 786	-45105 233	-12161 205	-2 62045E6	-475 22828E3	10 89531E6
	2 THD	984 477	-2751 468	-106 920	-17135 721	-1902 548	-259 83958E3
2804	6 DL	287 015	28212 750	-172 211	-27620 480	2597 810	-58930 926
	7 LL	124 049	9421 301	-72 843	-11686 514	1121 855	-25645 814
	1 MODAL LOA	-48953 379	126 48022E3	-36255 500	-8 74738E6	-256 65186E3	14 99306E6
	2 THD	2953 012	-690 461	-3 316	-668 033	-9783 059	-572 34000E3
2806	6 DL	286 497	28204 351	172 528	27684 418	-2559 470	-58829 496
	7 LL	124 087	9422 063	72 818	11682 038	-1124 031	-25653 025
	1 MODAL LOA	-45554 925	-172 81020E3	-36517 302	-8 78934E6	-548 35687E3	14 14661E6
	2 THD	2953 566	-600 158	0 334	-81 452	9718 572	-572 45625E3
2808	6 DL	-287 091	28234 379	-172 315	-27643 393	-2624 434	59292 270
	7 LL	-124 017	9425 974	-72 757	-11672 561	-1141 398	25838 803
	1 MODAL LOA	-45027 458	119 76204E3	-36242 809	-8 74958E6	553 86519E3	14 03773E6
	2 THD	2979 583	676 474	-0 476	-252 194	-9912 042	-577 61513E3
2810	6 DL	-287 060	28233 258	172 339	27647 904	2623 924	59286 855
	7 LL	-124 019	9425 876	72 754	11671 736	1141 788	25839 275
	1 MODAL LOA	-48429 040	-179 97658E3	-36622 596	-8 81057E6	252 64325E3	14 88521E6
	2 THD	2979 313	661 969	-3 390	-720 686	9944 321	-577 55819E3
2906	6 DL	2417 517	14549 035	-381 247	-61109 895	-6833 634	-374 14500E3
	7 LL	1076 325	3985 551	-162 205	-26000 719	-3043 263	-166 51909E3
	1 MODAL LOA	-29694 599	17390 728	-11454 928	-2 50726E6	488 61700E3	11 58661E6
	2 THD	987 358	2748 723	-107 380	-17265 531	1894 375	-261 32288E3
2908	6 DL	2417 513	14549 295	381 285	61119 277	6834 918	-374 14384E3
	7 LL	1076 324	3985 572	162 211	26002 287	3043 306	-166 51894E3
	1 MODAL LOA	-32319 723	-55161 924	-12744 238	-2 71436E6	337 23159E3	12 31756E6
	2 THD	987 208	2741 664	105 914	16898 467	-1852 399	-261 28309E3

2.7 Detailed Analysis of Strain Energy, Damping Energy, and Composite Damping for Each Mode

Mo de	Strain energy (Kn-m)	Damping energy (Kn-m)	Composite damping
1	1.405651E+00	6.993234E-02	0.0498
2	1.351426E+00	6.732269E-02	0.0498
3	1.361154E+01	6.770018E-01	0.0497
4	1.156962E+01	5.775057E-01	0.0499
5	5.077401E+00	2.475036E-01	0.0487
6	4.564975E+00	2.256239E-01	0.0494
7	1.690700E+01	6.934086E-01	0.0410
8	1.650246E+01	6.907169E-01	0.0419
9	2.323495E+01	1.156028E+00	0.0498

Table 3: Mode-wise Strain Energy, Damping Energy, and Composite Damping

This table reflects the dynamic behavior of a two-decked cable-stayed concrete bridge across different vibration modes. Modes 1 and 2 have low strain and damping energy (around 1.35–1.41 kN·m and 0.067–0.07 kN·m), indicating gentle, flexible movements typical of the structure's basic response. Modes 5 and 6 fall in a moderate range, showing slightly higher energy levels (about 4.56–5.08 kN·m for strain and 0.225–0.247 kN·m for damping), likely caused by localized deck activity. The higher modes—3, 4, 7, 8, and 9—show significantly larger strain and damping energy values (11.56–23.23 kN·m and 0.577–1.156 kN·m), suggesting more complex behavior involving the interaction of decks, cables, and towers. Interestingly, although Modes 7 and 8 carry high energy, their lower damping ratios (around 0.041–0.0419) hint at reduced efficiency in dissipating that energy.

2.7 Modal Weight Distribution and Directional Mass Participation per Mode

Modes	X - direction	Y - direction	Z - direction	Modal mass in kN
1	5.20 9566 E-09	2.28 5243 E-07	1.720 182E +05	4.026312 E+04
2	9.24 3531 E-05	1.88 8321 E-08	3.930 980E +02	3.402348 E+04
3	4.54 8439 E+05	1.72 0857 E-04	7.990 181E- 06	2.798438 E+05
4	1.58 3164 E-06	2.90 0987 E-06	1.443 893E +05	1.453194 E+05
5	3.52 4343 E+00	1.02 7369 E+05	1.525 020E- 05	5.079304 E+04
6	7.60 7954 E+02	1.10 5899 E+00	5.552 872E- 06	4.495290 E+04
7	9.54 3631 E+01	7.75 9756 E+03	5.012 131E- 06	1.545850 E+05
8	8.52 9245 E+03	2.95 1263 E+02	7.886 593E- 06	1.378927 E+05
9	1.05 0452 E-03	2.42 7832 E-04	8.624 829E +01	1.924801 E+05

Table 4: Mode-wise Distribution of Modal Weights and Generalized Mass Participation in X, Y, and Z Directions

This table shows the modal weights (calculated as modal mass times gravitational acceleration) for the first nine vibration modes of a two-decked cable-stayed concrete bridge. The values, given in kilo newtons (kN), represent the distribution of weight in the X, Y, and Z directions, as well as the total generalized weight for each mode. Mode 3 has the highest generalized weight at 279,843 kN, with the majority of this weight concentrated in the X-direction (454,843 kN), highlighting its significant contribution to the bridge's dynamic behavior. Mode 1, with a generalized weight of 40,263 kN, shows a major influence from the Z-direction (172,018 kN), indicating its importance in vertical motion. Mode 8, with a total weight of 137,893 kN, demonstrates a more balanced

distribution between the X- direction (8,529 kN) and Y-direction (295 kN), suggesting a more even involvement in horizontal movements. On the other hand, Mode 9, with a generalized weight of 192,480 kN, has minimal participation from the Y-direction (242 kN), indicating less influence in this mode.

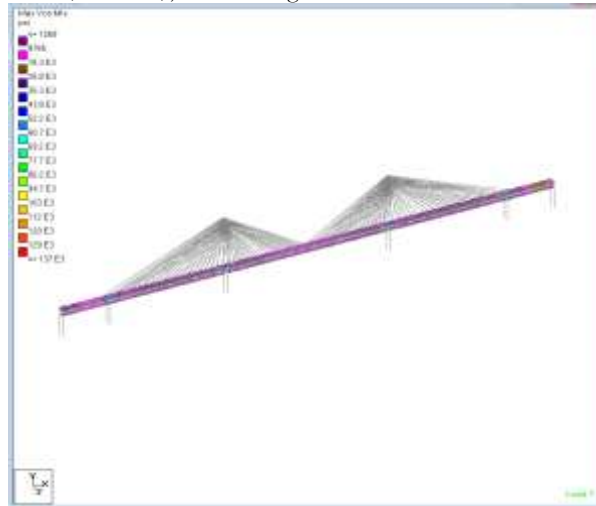


Fig: 7 shown in fig The stress on plate element

2.8 Mass Participation Factors for Each Mode in X, Y, and Z Directions

Mode	X-direction	Y-direction	Z-direction	SUM M-X	SUM M-Y	SUM M-Z
1	0	0	36.53	0	0	36.527
2	0	0	0.08	0	0	36.61
3	96.57	0	0	96.574	0	36.61
4	0	0	30.66	96.574	0	67.270
5	0	16.31	0	96.574	16.127	67.270
6	0.16	0	0	96.736	16.127	67.270
7	0.02	1.22	0	96.736	17.345	67.270
8	1.81	0.05	0	98.567	17.391	67.270
9	0	0	0.02	98.567	17.391	67.289

Table 4: Mode-wise Mass Participation Factors

3. Results:

1. This investigation examines the vibrational characteristics and dynamic response of a dual-deck cable-stayed concrete bridge utilizing STAAD.Pro. The time-history analysis identified peak base shear forces at 1.124992 seconds, recorded as follows: X-direction = 27,089.90 kN, Y-direction = 345.83 kN, and Z-direction = -10.53 kN.
2. The dynamic behavior of a two-decked cable-stayed concrete bridge shows varying strain and damping energy across modes, with low energy in modes 1 and 2 (strain: 1.35–1.41 kN•m, damping: 0.067–0.07 kN•m), moderate energy in modes 5 and 6 (strain: 4.56–5.08 kN•m, damping: 0.225–0.247 kN•m), and high energy in modes 3, 4, 7, 8, and 9 (strain: 11.56–23.23 kN•m, damping: 0.577–1.156 kN•m), with modes 7 and 8 showing lower damping efficiency (0.041–0.0419).
3. The modal weight analysis of the two-decked cable-stayed concrete bridge reveals that Mode 3 plays the most significant role, with a total generalized weight of 279,843 kN, driven mainly by a large contribution in the X-direction (454,843 kN). Mode 1 comes next, with a weight of 40,263 kN, showing substantial vertical motion (172,018 kN in the Z-direction). Mode 8 has a more balanced weight distribution, totaling 137,893 kN, with contributions from both the X-direction (8,529 kN) and Y-direction (295 kN). Finally, Mode 9 has a generalized weight of 192,480 kN, but its contribution from the Y-direction (242 kN) is minimal, indicating it has a smaller impact on the overall behavior of the bridge.
4. The modal analysis of the two-decked cable-stayed concrete bridge shows dominant X-direction mass participation from Modes 3–9 (96.574%–98.567%), minimal input from Modes 1–2, Y-direction influence mainly from Modes 5 (16.13%) and 7 (1.22%), and Z-direction dominance by Modes 1 (36.53%) and 4 (30.66%), highlighting distinct directional contributions across modes.
5. Maximum displacement recorded is approximately 0.688 m in the vertical direction at Node 2.

3.1 CONCLUSION:

1. Dynamic Response Identified: The Bridge shows distinct vibrational behavior, with Mode 3 contributing most significantly to horizontal movement, while Modes 1 and 4 influence vertical dynamics.
2. Base Shear Peaks Early: The highest base shear occurs at 1.12 seconds, with the X-direction experiencing the greatest force, emphasizing the need for strong lateral stability.
3. Critical Node Displacement: Maximum vertical displacement of approximately 0.688 m was observed at Node 2, highlighting it as a key area for monitoring and design focus.
4. Damping Is Essential: Damping effectively reduces vibration amplitudes over time, playing a major role in enhancing the bridge's ability to handle repeated dynamic loading.
5. Design Implications: These findings provide valuable guidance for designing safer and more resilient double-deck cable-stayed bridges, especially in space-constrained, high-traffic regions.

3.2 Areas of future exploration

1. Exploring Other Materials: Future work could look into using different materials like steel, composites, or high-performance concrete to see how they affect the bridge's vibration behavior, damping capacity, and overall performance.
2. Tweaking Material Properties: Even with the same material—like concrete—changing its properties such as stiffness, density, or damping could help fine-tune the bridge's response to dynamic loads.
3. Trying New Configurations: Adjusting the shape and size of structural elements—like increasing pylon height, changing deck thickness, or rearranging cables—may reveal ways to improve the bridge's strength and stability.
4. Using Hybrid Designs: Combining materials, such as using a steel-concrete hybrid system, could enhance strength while reducing weight, offering a smarter approach to bridge design.
5. Pretending More Realistic Loads: By testing how the bridge reacts to different forces—like earthquakes, strong winds, or heavy traffic—future studies can better predict how it would behave in real-world conditions.

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