

Seismic Response of Long-Span Arch Bridges Under Bhuj Ground Motion: Structural-Form, Span, and Topography-Oriented Assessment

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Abstract

Long-span arch bridges are frequently selected for valley crossings because compression-dominated arch action provides high structural efficiency, yet their large spatial extent and concentrated springing forces make their earthquake response sensitive to structural form, span, support conditions, and terrain-related irregularity. This study evaluates the reported seismic behaviour of straight and arch bridge configurations and establishes a topography-oriented modelling framework without introducing data beyond the uploaded report. Three-dimensional models were developed in STAAD.Pro V8i using M30 concrete, a 168 m reference length, dead load, IRC Class AA moving load, and the Bhuj 2001 ground-motion record. Seven configurations were considered: straight and arch bridges of 168 m and 84 m and arch models with 20 m, 30 m, and 40 m deck spans. Static validation used an 80 m deck-and-girder sub-model and compared manual bending moments and shear forces with numerical output. The inner-girder bending moment showed close agreement, with 3360 kN-m manually and 3387 kN-m numerically. For the 168 m models, arch action reduced displacement from 21.8 to 13.8 mm, acceleration from 7.68 to 4.61 mm/s², velocity from 14.72 to 8.83 mm/s, and maximum shear from 9.92 to 5.952 kN, corresponding to reductions of about 36.70–40.01%. Arch-span displacement increased from 13.1 mm at 20 m to 30.5 mm at 40 m. The results demonstrate the benefit of arch action, the flexibility penalty of increasing span, and the need to extend the validated model to flat, shallow, deep, and asymmetrical valley conditions using soil–structure interaction and spatially varying support excitation.

Keywords—*Arch bridge; Bhuj earthquake; finite-element analysis; seismic response; STAAD.Pro; time-history analysis; topography.*

1. INTRODUCTION

Long-span arch bridges are essential links across rivers, gorges, and mountainous corridors, where their curved load path transfers a substantial proportion of gravity loading as compression toward the springings. This structural efficiency permits long unobstructed spans, but it also concentrates axial thrust, bending, and shear near the arch feet and creates dynamic behaviour that differs from conventional straight-girder systems. Earthquake response becomes especially complex when a bridge crosses irregular terrain, because ridges, slopes, and valleys can reflect, refract, scatter, and diffract seismic waves, thereby modifying their amplitude, frequency content, duration, and phase [1]. Recent syntheses further show that the magnitude of topographic amplification depends jointly on landform geometry, incident-wave characteristics, and subsurface stiffness [2]. For a spatially extended bridge, the assumption of identical motion at every support may therefore be unrealistic: wave-passage delay, loss of coherence, and differential site response can generate asynchronous support movements [3]. Nonuniform support excitation can redistribute internal forces and shift critical response locations in large-span arch bridges [4]. Accordingly, seismic assessment must consider not only global deck displacement but also arch-crown movement, springing reactions, axial-force variation, bending moment, shear, acceleration, and stress at critical sections. This broader response set is necessary because local demand may govern even when deck motion is modest.

The engineering importance of this problem is heightened by the low-frequency modes, large support spacing, heavy deck systems, and multidirectional response of long-span bridges. Near-field and far-field records can produce markedly different demand patterns because long-period pulses interact with flexible global modes and alter axial-force–moment combinations in reinforced-concrete arches [5]. Damage-oriented investigations of bridge–track systems likewise indicate that relative displacement between adjacent components may govern local vulnerability even when absolute displacement remains moderate [6]. Seismic-control studies have identified bearings, transverse connections, arch feet, and bracing systems as important performance locations, demonstrating that stiffness distribution and energy-dissipation

strategy influence both displacement and force demand [7]. Practical finite-element analysis in STAAD.Pro offers an accessible route for integrating geometry, material properties, moving loads, support conditions, modal analysis, and ground-motion histories [8]. The Rudramata Bridge near Bhuj provides a relevant case context because its 168 m length, reinforced-concrete A-frame piers, and exposure to the 2001 Bhuj earthquake establish a realistic basis for comparing structural configurations [9]. However, a bridge model placed on level supports cannot, by itself, reproduce the terrain-related irregularity commonly associated with canyon crossings. Consequently, site representation must accompany structural idealisation when bridge performance is judged for mountainous alignments.

Existing analysis procedures frequently simplify earthquake demand through a global seismic coefficient, a uniform response spectrum, or a single acceleration history imposed simultaneously at all supports. Such approaches are useful for preliminary design, but they cannot fully represent asynchronous support movement or the response changes caused by unequal foundation elevations, variable pier heights, asymmetric slopes, or local stiffness differences. Earlier bridge studies often examined either structural form, span, near-fault motion, spatial variability, or topographic amplification in isolation. Free-field topography studies rarely transferred modified motions into a detailed arch-bridge model, while many arch-bridge investigations applied records without explicitly modelling the valley that motivated the bridge form. This separation makes it difficult to distinguish the structural effect of arch action from the site effect of irregular terrain. The conventional seismic-coefficient expression and response-spectrum basis used in the source report follow the earlier national seismic design framework [10], whereas the time-history comparison preserves one common record to avoid introducing record-to-record variability. The principal problem is therefore to determine how arch action and span affect response under the Bhuj record and to establish the validated basis required for flat, shallow-valley, deep-valley, and asymmetric-valley extensions. A controlled comparison is necessary to avoid attributing site-related effects incorrectly to span or structural form.

The study develops and validates three-dimensional straight and arch bridge models in STAAD.Pro, applies dead load, IRC Class AA moving load, and the Bhuj 2001 acceleration history, and extracts displacement, acceleration, velocity, and shear response. Static credibility is checked by comparing manually calculated and software-derived bending moments and shear forces for an 80 m validation sub-model. Seven configurations are assessed: straight and arch bridges of 168 m and 84 m, together with 20 m, 30 m, and 40 m arch-span models. The original contribution is a consolidated assessment that links structural-form comparison, span sensitivity, manual finite-element validation, and a topography-oriented variable framework without inventing terrain-response results not contained in the source report. The paper first reviews 15 directly relevant studies; it then presents the methodology, modelling assumptions, geometry, materials, loading, boundary conditions, and validation procedure. Subsequent sections interpret the numerical comparisons, quantify response reductions and span-related increases, identify the best-performing reported configuration, and explain the engineering implications. The conclusion summarises the achieved objectives, while the future scope defines the additional terrain, soil–structure interaction, and multi-support analyses needed for a complete topographic seismic assessment. This organization maintains a clear distinction between completed findings and proposed analytical extensions.

2. LITERATURE REVIEW

Li et al. modelled obliquely incident P and SV waves at a V-shaped canyon and found incidence angle and canyon geometry to govern asymmetric arch-bridge demand, with vulnerable abutment and quarter-span regions [11]. Zhang et al. generated canyon-compatible motions for a deep-water rigid-frame bridge and showed that flat-site uniform input can underestimate foundation and pier response [12]. Li et al. used OpenSees to demonstrate stronger demand in deep canyons and on illuminated slopes, including bearing residual deformation [13]. Zhu et al. reported nearly doubled adjacent-pier relative displacement in an irregular railway bridge–track system [14]. Zhang et al. developed an analytical canyon–bridge interaction solution for oblique SH waves, confirming amplified response at the wave-facing foundation [15]. Li et al. distinguished amplification and shielding across opposing slopes [16]. Liu et al. coupled boundary- and finite-element methods to simulate stochastic valley motions, sediment, water, and reduced coherence [17]. Wang et al. showed that oblique P-wave incidence changes support phase, horizontal–vertical coupling, and internal forces [18]. Together, these studies justify spatially consistent support input, although most use simplified or non-arch systems.

Wang et al. found that sedimentary canyon thickness and stiffness alter motion amplitude and frequency content through resonance and interference [19], while their fragility framework identified peak ground velocity as an efficient intensity measure and showed that topography can change the most vulnerable component [20]. Álvarez et al. established that

earthquake-induced axial-force fluctuation modifies arch-section capacity and must be assessed with bending demand [21]. Liu et al. used correlated near-fault multi-station records for a steel-truss arch bridge and demonstrated that uniform excitation cannot reproduce asymmetric damage patterns [22]. Ye et al. evaluated friction-pendulum bearings and stayed-buckle cables, finding the combined strategy most effective [23]. Zhang et al. developed a MATLAB–ANSYS parametric model and identified arch feet, crown, transverse stiffness, and multidirectional input as critical [24]. Xia et al. showed that near-fault pulses and excitation angle strongly influence tied-arch damage [25]. These studies provide advanced mechanisms and mitigation insights, but none supplies the controlled STAAD.Pro comparison reported here. The present work addresses that gap through common geometry, Bhuj input, static validation, quantified form and span comparisons, and a defined route toward terrain-specific modelling.

3. RESEARCH METHODOLOGY

The analytical programme began with identification of the mismatch between long-span arch bridges constructed in valleys and the common use of level supports with uniform excitation. Source-report geometry, material data, loading, and earthquake parameters were collected and organised into a controlled model matrix. Traffic actions and impact provisions were based on IRC:6-2010 [26], concrete bridge design requirements on IRC:112-2011 [27], seismic zoning and response parameters on IS 1893 (Part 1):2016 [28], reinforcing-steel specification on IS 1786:2000 [29], and bridge carriageway provisions on IRC:5-1998 [30]. The same principal input values were maintained across comparable models so that response differences could be attributed to structural form or span rather than inconsistent loading.

The modelling sequence comprised preparation of straight and arch geometries, assignment of beam and plate properties, support specification, application of dead load and IRC Class AA moving load, and definition of the Bhuj 2001 time-history function. Modal analysis was used to examine the first six global deformation patterns, and time-history analysis generated displacement, acceleration, velocity, and member-force response. The reported study used a uniform ground-motion input for its numerical comparisons. Topographical influence was defined as a subsequent controlled extension through flat, shallow symmetrical, deep symmetrical, and asymmetrical valley profiles, with support elevation, pier height, slope, and foundation restraint treated as variables.

Validation was performed independently of the 168 m dynamic model. An 80 m deck-and-girder sub-model reproduced the material properties, support conditions, and loads used in the manual calculations. Dead-load and live-load bending moments and shear forces were calculated for inner and outer girders and compared with STAAD.Pro output. After verification, the response of seven analytical configurations was extracted and compared. Percentage reduction for the arch model was calculated relative to the corresponding straight model, whereas span-related change was calculated relative to the 20 m arch-span case. Final performance evaluation considered the magnitude of global responses and the consistency of observed trends with the arch load-transfer mechanism.

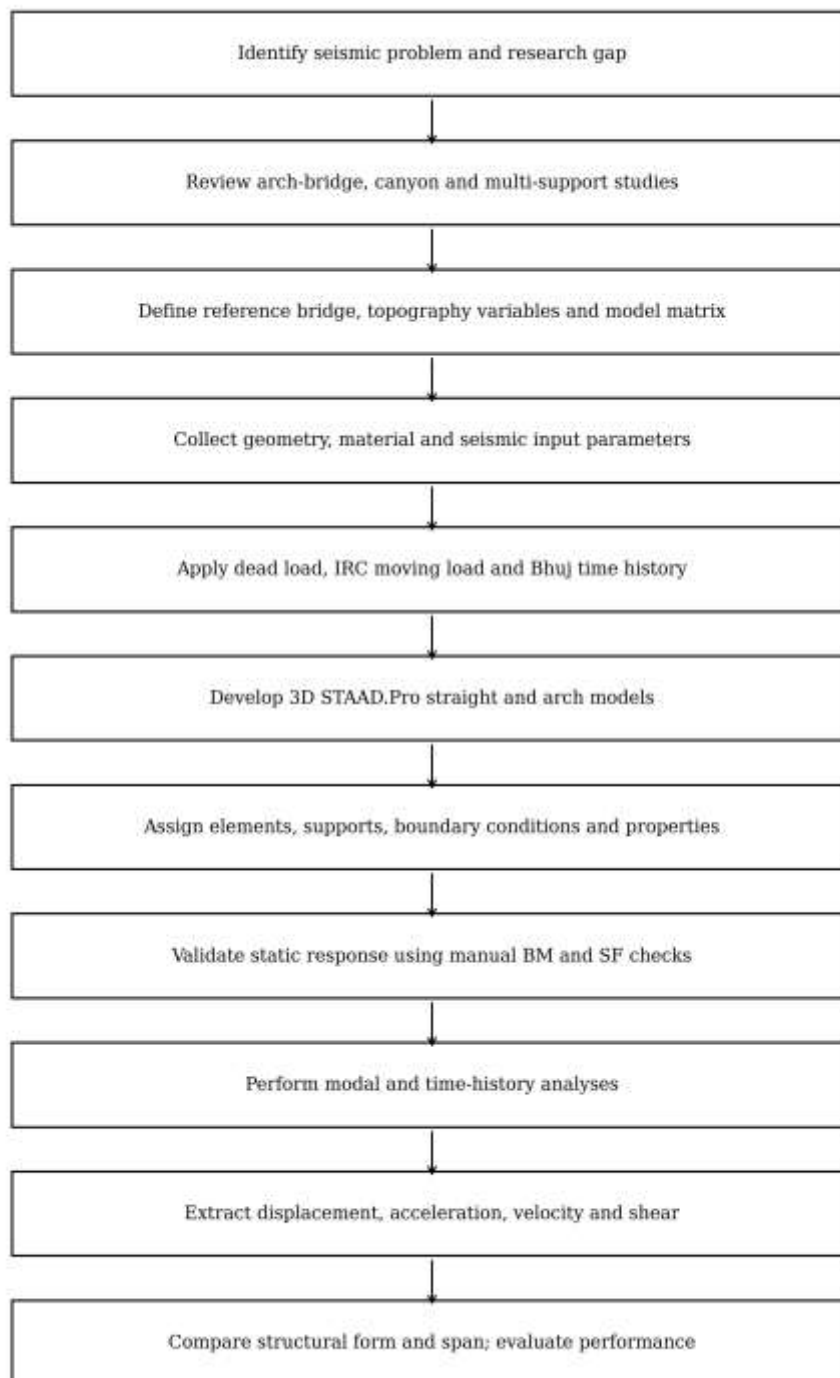


Figure 1. Research methodology flowchart

The flowchart presents the complete source-based sequence from problem identification to final performance assessment. It deliberately separates data definition, modelling, validation, analysis, and comparison so that the numerical results can be traced to controlled inputs. The inclusion of manual validation before dynamic interpretation is significant because it checks the basic transfer of gravity and moving loads. Topography is introduced as a defined modelling extension rather than as a completed result set, thereby preventing unsupported claims of terrain amplification.

Table 1. Research variables and comparison framework

Category	Variable or measure	Role in the study
Independent	Structural form; span; topographical configuration	Compares straight and arch systems, evaluates 20–40 m span variation, and defines flat and valley extensions.
Controlled	Geometry, materials, loading, damping, Bhuj record, target intensity	Maintains a consistent basis for valid model-to-model comparison.
Dependent	Displacement, acceleration, velocity, shear; proposed local forces and stress	Quantifies global dynamic response and defines outputs for future topography-sensitive analysis.
Topographic	Valley depth H , width B , H/B , slope angle θ , support elevations EL and ER	Represents terrain severity, symmetry, unequal pier height, and foundation-level variation.
Comparison	Absolute response and percentage change	Expresses reductions relative to the straight bridge and increases relative to the 20 m span.

Table 1 distinguishes the variables actually quantified in the reported analysis from the topographical variables defined for later implementation. Structural form and span are the completed numerical comparisons, while valley geometry and support elevations form the independent site parameters for future models. This separation is essential because the source report does not provide numerical response values for the four valley profiles. The controlled-variable strategy supports causal interpretation by holding materials, loading, earthquake record, and analysis procedure constant.

4. MODELLING AND DESIGN

The case-study context is the Rudramata Bridge near Bhuj, constructed in 1966 with ten simply supported spans of 16.8 m and an overall length of 168 m. The bridge is approximately 7.3 m wide and is supported on reinforced-concrete A-frame piers with elastomeric bearings. The analytical work did not attempt to reproduce every field detail; instead, it used the reported dimensions and common material and loading parameters to establish comparable straight and arch configurations. A separate 80 m deck-and-girder model was retained solely for static validation and should not be interpreted as the same geometry as the principal dynamic model.

Table 2. Geometrical specifications

Parameter	Principal case-study model	Validation sub-model
Total length	168 m (10×16.8 m)	80 m
Bridge width / carriageway	7.3 m; two lanes	Deck-and-girder section used for manual checking
Main girders	10	Longitudinal and cross-girder system
Average slab thickness	0.30 m	0.25 m
Longitudinal girder	As assigned in STAAD.Pro	400 mm wide $\times$ 1400 mm clear depth
Cross girder	As assigned in STAAD.Pro	300 mm wide $\times$ 1400 mm clear depth
Wearing coat	Included in dead load	80 mm
Additional arch spans	20 m, 30 m, and 40 m	-

The principal and validation geometries serve different purposes. The 168 m system establishes the structural-form comparison and the Rudramata case context, whereas the 80 m sub-model provides a manageable manual-design check. Retaining this distinction prevents the validation results from being misapplied to the long-span dynamic model. The

0.30 m principal slab and 0.25 m validation slab are reported separately; no attempt is made to reconcile them because each belongs to a different analytical model.

Table 3. Material properties used in the numerical models

Property	Adopted value	Modelling significance
Concrete grade	M30	Defines the reinforced-concrete bridge members.
Compressive strength, f_{ck}	30 MPa (30,000 kN/m ² as reported)	Basis for stiffness and section design parameters.
Elastic modulus, E	27,386.128 MPa (27,386,128 kN/m ²)	Controls elastic member and plate stiffness.
Poisson's ratio	0.18	Defines transverse elastic strain response.
Unit weight / density	25 kN/m ³	Generates self-weight and seismic mass.
Reinforcement	High-strength deformed bars to IS 1786	Provides the stated reinforcement specification basis.

Table 3 summarises the material inputs explicitly available in the report. M30 concrete with an elastic modulus of 27,386.128 MPa and Poisson's ratio of 0.18 was used consistently, while the 25 kN/m³ unit weight generated dead load and mass. The reported analysis is therefore treated as elastic unless specific nonlinear constitutive laws, hinges, links, or degradation rules are assigned. The source report cautions that geometric nonlinearity must not be described as material nonlinearity.

Table 4. Loading and seismic input parameters

Parameter	Adopted value	Application
Dead load	Self-weight, slab, girders, wearing coat and permanent components	Establishes gravity state and seismic mass.
Live load	IRC Class AA train / vehicle loading	Applied as moving load on the deck.
Impact allowance	As per IRC 6; 18% used in the slab design check	Accounts for dynamic vehicle effect.
Seismic record	Bhuj earthquake, 2001	Common time-history input for all reported models.
Arrival time	1 s	Input parameter reported for the time-history definition.
Seismic zone and factor	Zone III; $Z = 0.16$	Defines regional seismic parameter in the report.
Response reduction factor	$R = 3$	Adopted software input.
Soil type	Type II	Reported site-class assumption.
Load combination	DL + LL + EQ (time history)	Primary comparative loading condition.

The loading table confirms that every reported bridge model was subjected to the same gravity, traffic, and earthquake basis. Using one Bhuj acceleration record isolates the effect of structural form and span, although it does not capture record-to-record uncertainty. Zone III, $Z = 0.16$, $R = 3$, and Soil Type II are retained exactly as reported. The impact allowance is relevant to the static deck check, whereas the time-history response is governed by the common seismic function and model properties.

Table 5. Boundary conditions and modelling details

Item	Reported modelling approach
Software	STAAD.Pro V8i
Deck slab	Plate / shell finite elements
Girders, piers and arch members	Beam elements
Supports	Specified through the STAAD.Pro support page; fixed or equivalent restraints where adopted
Foundation / soil	Fixed support or equivalent springs when data are available
Dynamic procedure	Modal analysis followed by time-history analysis
Meshing	Plate discretisation shown in the validation model; exact mesh size not reported
Nonlinearity	No material nonlinearity claimed in the reported result set

The modelling details reveal a conventional three-dimensional frame-and-plate idealisation. Beam elements carry the principal member actions, while plate elements distribute deck loading. The report does not state a numerical mesh size, damping ratio, or detailed nonlinear material model; therefore, these values are not inferred. Boundary conditions were maintained consistently within each comparison. Future topography models should replace uniform fixed restraints with elevation-dependent supports or calibrated springs when reliable site data are available.

Table 6. Analytical model configurations

Model	Configuration	Primary comparison purpose
1	Straight bridge, 168 m	Reference for principal long-span form comparison.
2	Arch bridge, 168 m	Evaluates the effect of arch action at equal overall length.
3	Straight bridge, 84 m	Shorter straight-bridge response.
4	Arch bridge, 84 m	Shorter arch-bridge response.
5	Arch bridge with 20 m deck span	Baseline for span-variation comparison.
6	Arch bridge with 30 m deck span	Intermediate span sensitivity.
7	Arch bridge with 40 m deck span	Longest reported parametric span.

The seven-model matrix supports two independent comparisons. Models 1–4 examine structural form at 168 m and 84 m, while Models 5–7 isolate arch-span variation. The 168 m arch and straight systems constitute the principal result pair because they share the case-study length and common input. The report also defines four terrain configurations conceptually, but their numerical results are not included; consequently, they are not added to this completed model matrix.

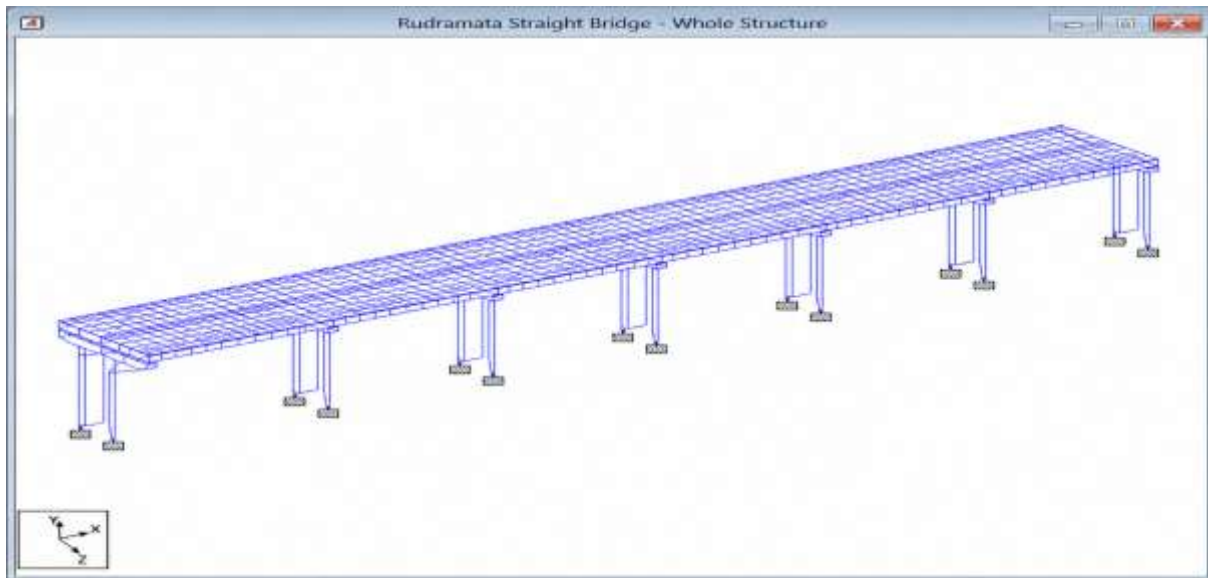


Figure 2. STAAD.Pro model of the 168 m straight bridge

The straight-bridge model shows the longitudinal deck, repeated supports, and global finite-element discretisation used as the reference configuration. Its significance lies in providing an equal-length baseline against which arch action can be evaluated. Because the same concrete properties, load cases, and Bhuj input were applied to both principal models, the difference in response reflects the change in structural load path and stiffness rather than a change in excitation.

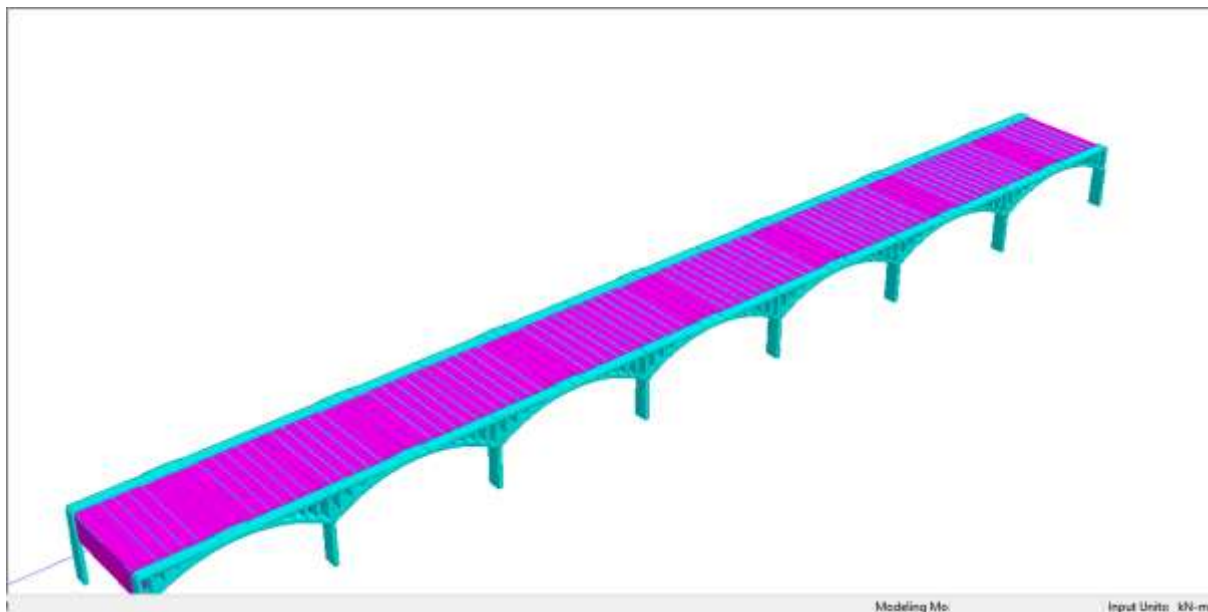


Figure 3. Typical STAAD.Pro model of the arch bridge

The arch model introduces a curved compression-resisting system beneath the deck while retaining the common analytical input. The geometry creates a direct thrust path toward the springings and increases global stiffness relative to the straight system. This structural mechanism explains why the reported arch configuration develops lower displacement, acceleration, velocity, and maximum shear. Critical local checks for future work should focus on the arch feet, crown, quarter-span regions, spandrel members, and support reactions.

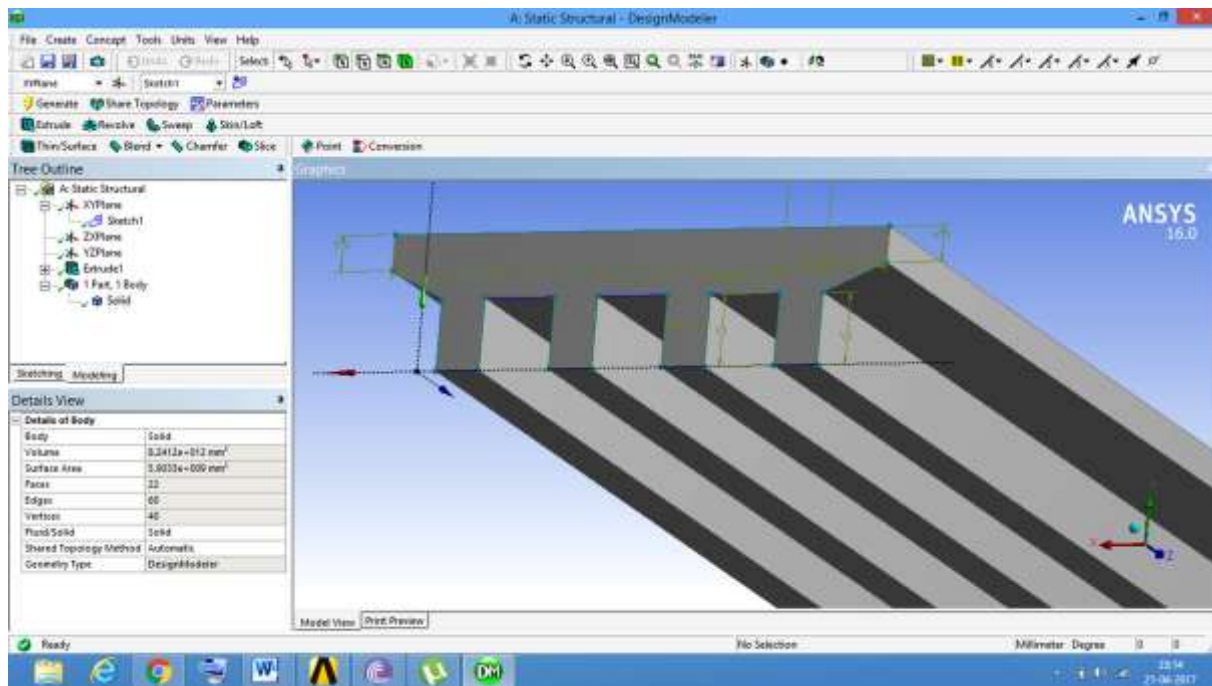


Figure 4. Bridge section and three-dimensional validation model

The validation figure presents the separate 80 m deck-and-girder system used to compare manual and finite-element actions. The model includes the slab, longitudinal girders, cross girders, and support arrangement needed to reproduce the selected loading case. Its role is not to validate every dynamic feature of the 168 m bridge, but to demonstrate that the adopted element idealisation and load transfer produce reasonable static bending moments and shear forces before seismic comparisons are interpreted.

5. RESULTS AND DISCUSSION

The reported numerical outputs comprise static validation, the first six mode-shape plots, displacement-time histories, a direct 168 m straight-versus-arch comparison, and a three-span arch comparison. The results do not contain numerical responses for flat, shallow-valley, deep-valley, or asymmetrical-valley models, nor do they provide nonlinear failure indices or detailed stress histories for the 168 m arch. Accordingly, the discussion quantifies only the available data and identifies topography-sensitive critical regions as a future analytical requirement rather than a completed finding.

Table 7. Manual and STAAD.Pro validation comparison

Response	Manual	STAAD.Pro	Difference	Percentage difference
Outer girder bending moment	4060 kN-m	3247 kN-m	-813 kN-m	-20.02%
Inner girder bending moment	3360 kN-m	3387 kN-m	+27 kN-m	+0.80%
Outer girder shear force	899 kN	784.68 kN	-114.32 kN	-12.72%
Inner girder shear force	899 kN	737 kN	-162 kN	-18.02%

The inner-girder bending moment provides the closest validation result: 3387 kN-m from STAAD.Pro is only 27 kN-m, or 0.80%, above the manual value of 3360 kN-m. Larger differences occur for the outer-girder moment and both shear-force comparisons, with software values 12.72–20.02% below the manual calculations. The source report attributes these differences to finite-element load distribution, support representation, and simplifying assumptions in the manual method. The overall agreement is adequate for comparative elastic assessment, but the discrepancies should be considered when interpreting local force demand.

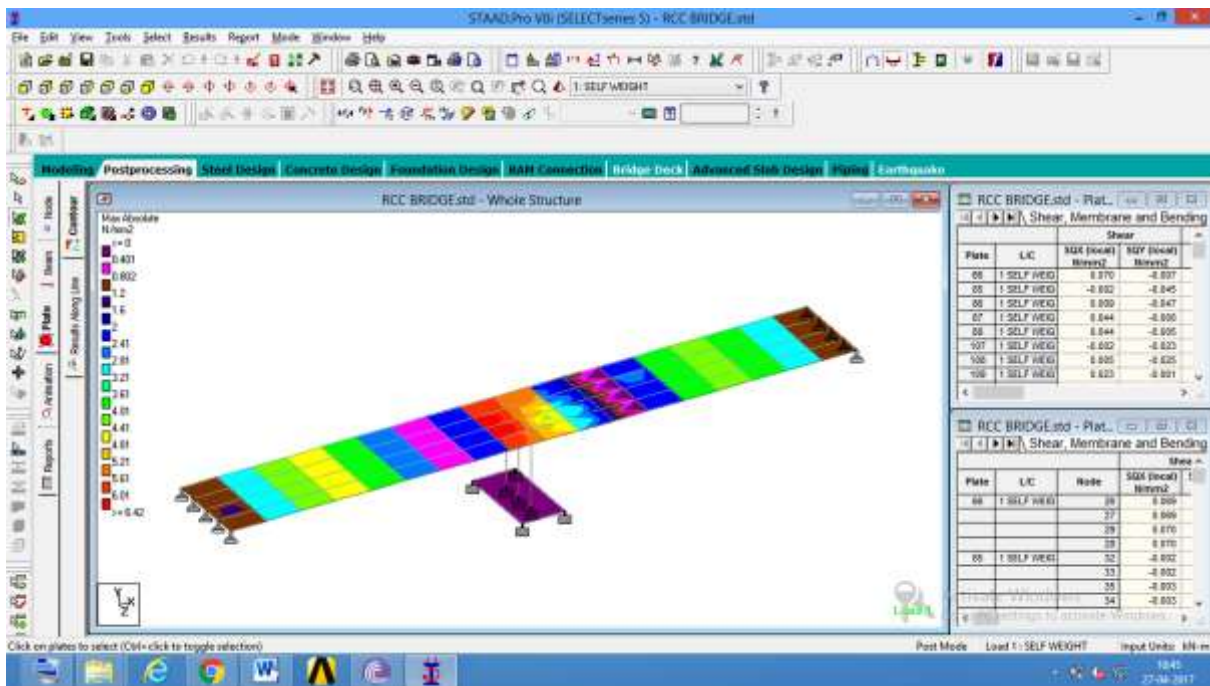


Figure 5. Finite-element stress variation in the validation deck

The stress plot illustrates the spatial distribution generated by the plate-and-beam idealisation rather than a single line-girder assumption. Variation across the slab and girders helps explain why finite-element bending moments and shear forces do not match simplified manual values exactly. The figure supports the validation interpretation that two-dimensional load distribution and local stiffness influence the numerical response. Quantitative stress magnitudes are not transcribed because the report does not provide a readable tabulated stress scale for this image.

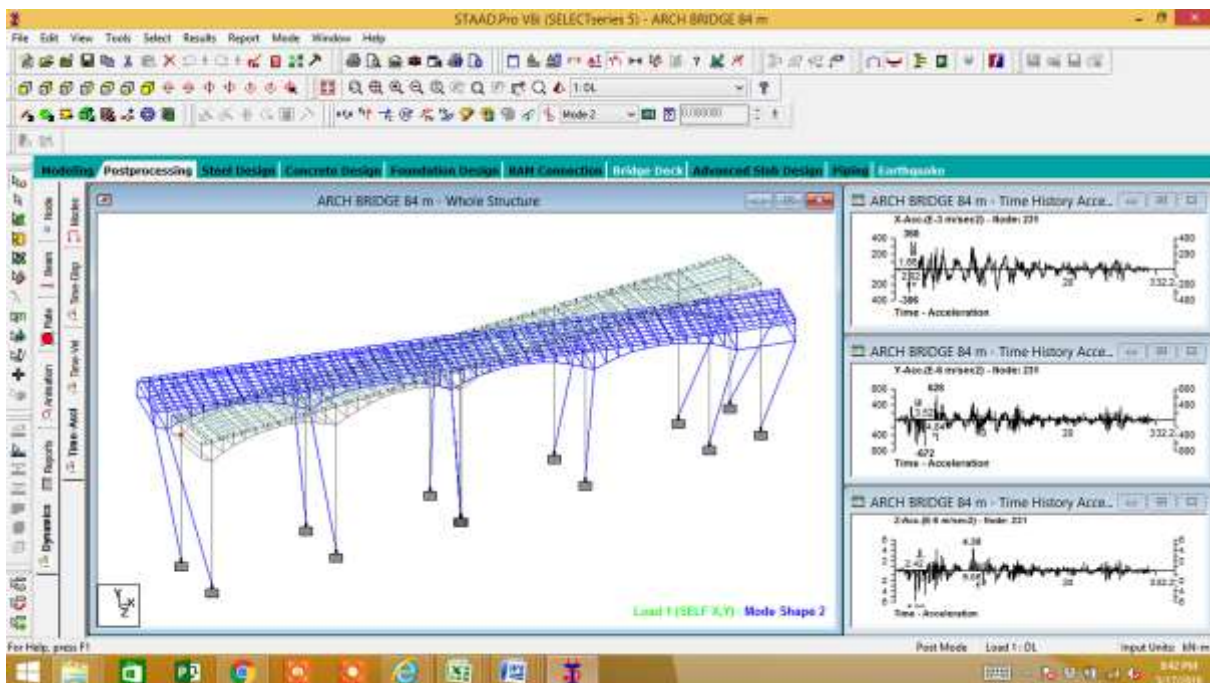


Figure 6. Representative global mode shape from the reported bridge analysis

The representative modal output confirms that the bridge response is spatially distributed and involves deformation of the deck and supporting system. The report presents six mode shapes, indicating that more than one vibration mechanism contributes to the seismic response. Although numerical periods and modal mass participation values are not tabulated, the sequence supports the use of a three-dimensional dynamic model. Future topographical analysis should verify sufficient cumulative mass participation and monitor possible translation–torsion coupling under asymmetrical support elevations.

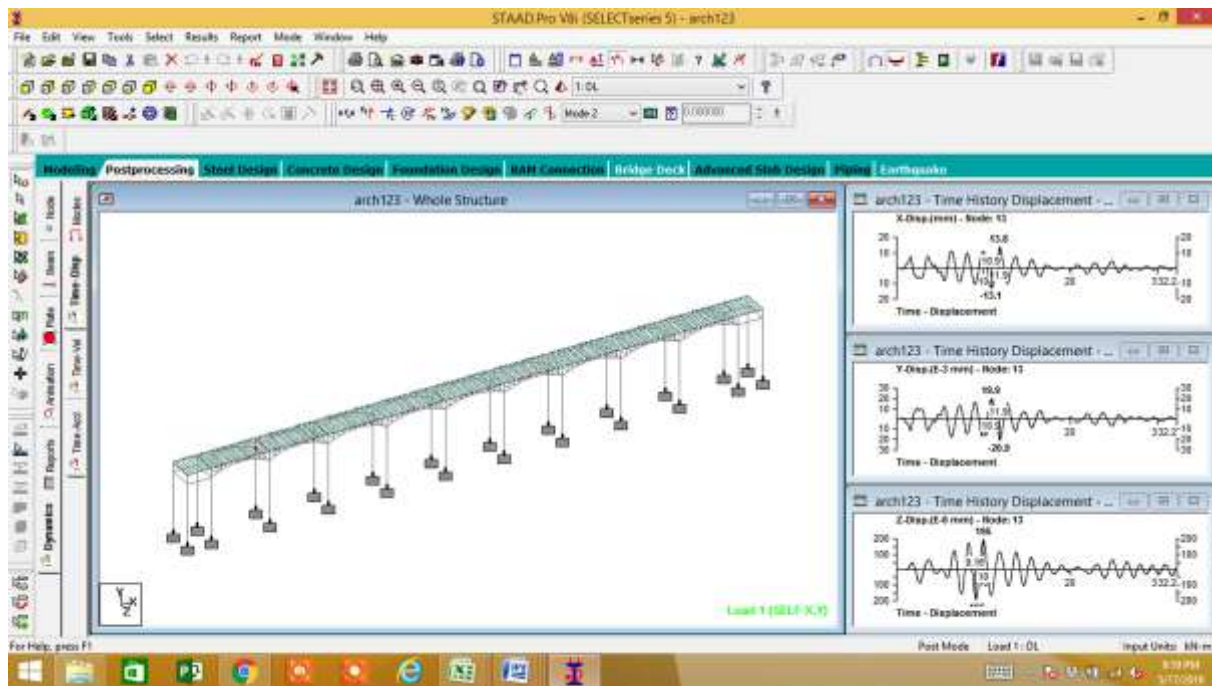


Figure 7. Displacement-time response of the 168 m arch bridge

The arch-bridge time-history output shows the evolution of displacement under the Bhuj acceleration function and demonstrates that peak response occurs during a limited portion of the record rather than under a static load pattern. Its maximum reported displacement is 13.8 mm. The relatively low peak compared with the straight bridge is consistent with the higher stiffness and compression-dominated load path of the arch system. The exact time of the peak is not tabulated in the report and is therefore not inferred.

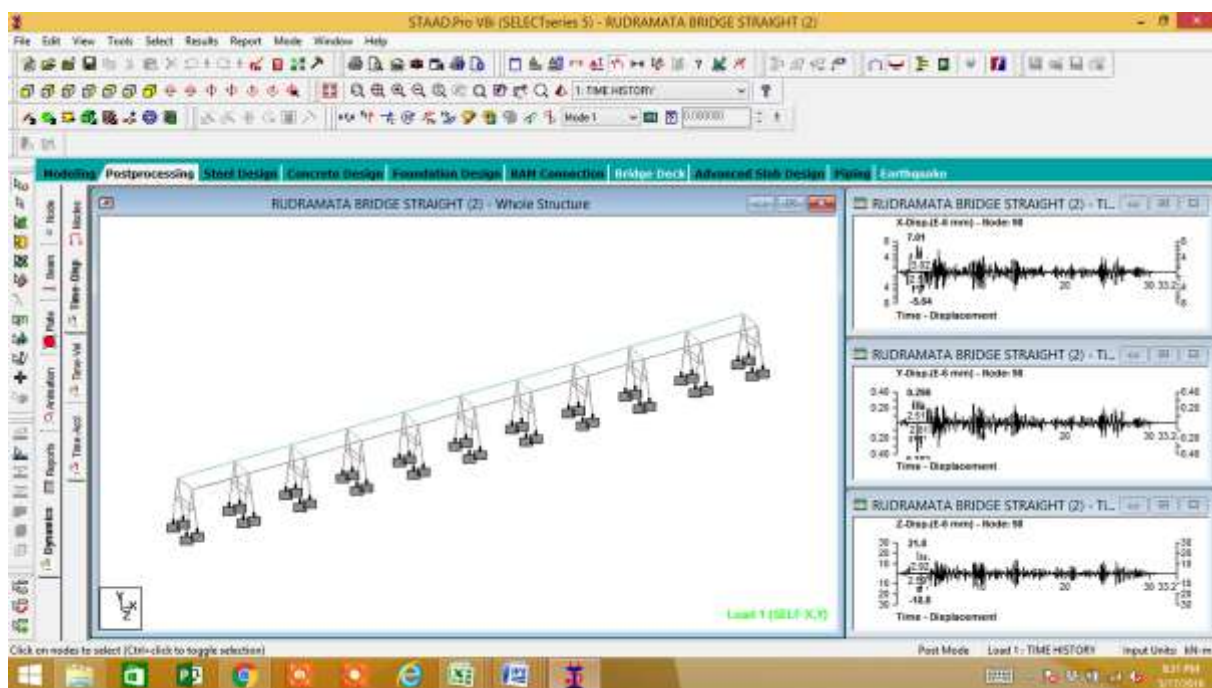


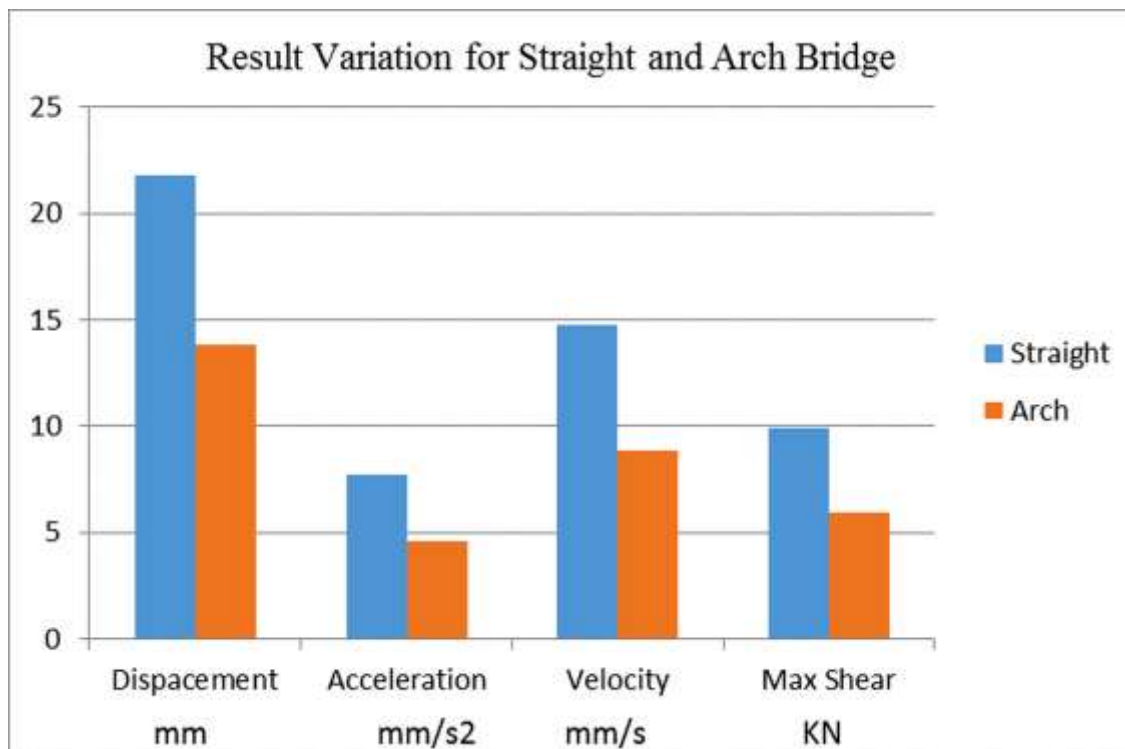
Figure 8. Displacement-time response of the 168 m straight bridge

The straight-bridge history reaches a reported maximum displacement of 21.8 mm, which is 8.0 mm greater than the 168 m arch response. The broader deformation demand reflects the comparatively flexible straight configuration under the same ground-motion record and loading basis. Comparing Figures 7 and 8 on a common input demonstrates the structural benefit of arch action; however, the histories do not quantify terrain-induced differential support movement because the reported excitation is uniform across supports.

Table 8. Time-history response of the 168 m straight and arch bridges

Parameter	Straight bridge	Arch bridge	Absolute reduction	Reduction
Displacement	21.8 mm	13.8 mm	8.0 mm	36.70%
Acceleration	7.68 mm/s ²	4.61 mm/s ²	3.07 mm/s ²	39.97%
Velocity	14.72 mm/s	8.83 mm/s	5.89 mm/s	40.01%
Maximum shear	9.92 kN	5.952 kN	3.968 kN	40.00%

The arch bridge is the best-performing reported 168 m configuration for every response parameter. Displacement falls by 36.70%, while acceleration, velocity, and maximum shear fall by approximately 40%. The close similarity of the latter three reductions indicates a consistent change in global dynamic demand rather than an isolated improvement in one metric. The lower displacement is attributed to the increased stiffness and direct compressive thrust path of the arch. These results support the use of arch systems for long crossings, subject to careful design of springings and foundations that resist horizontal thrust.


Figure 9. Comparative response of the 168 m straight and arch bridges

The comparative graph visually consolidates the four parameters listed in Table 8. In each category, the arch bar is below the straight-bridge bar, confirming a systematic response reduction. The largest absolute difference occurs in velocity, decreasing from 14.72 to 8.83 mm/s, while the largest relative difference is approximately 40.01%. The figure is interpreted using the tabulated values because those values provide the reliable numerical basis and avoid ambiguity from graphical scaling.

Table 9. Response of the arch bridge for span variation

Parameter	20 m	30 m	40 m	30 m vs 20 m	40 m vs 20 m
Displacement	13.1 mm	29.3 mm	30.5 mm	+123.66%	+132.82%
Acceleration	28.3 ×10 ⁻³ mm/s ²	30.3 ×10 ⁻³ mm/s ²	36.3 ×10 ⁻³ mm/s ²	+7.07%	+28.27%
Velocity	32.6 ×10 ⁻³ mm/s	43.4 ×10 ⁻³ mm/s	59.6 ×10 ⁻³ mm/s	+33.13%	+82.82%

Span increase produces a clear flexibility penalty. Displacement more than doubles between 20 m and 30 m and reaches 30.5 mm at 40 m, a 132.82% increase relative to the 20 m case. Velocity is also highly span-sensitive, increasing by 82.82% at 40 m. Acceleration rises more gradually, by 28.27% over the same range. The relatively small 4.10% displacement increase from 30 m to 40 m suggests that the major stiffness transition occurs between 20 m and 30 m for the reported configurations, although additional spans would be needed to define the trend robustly.

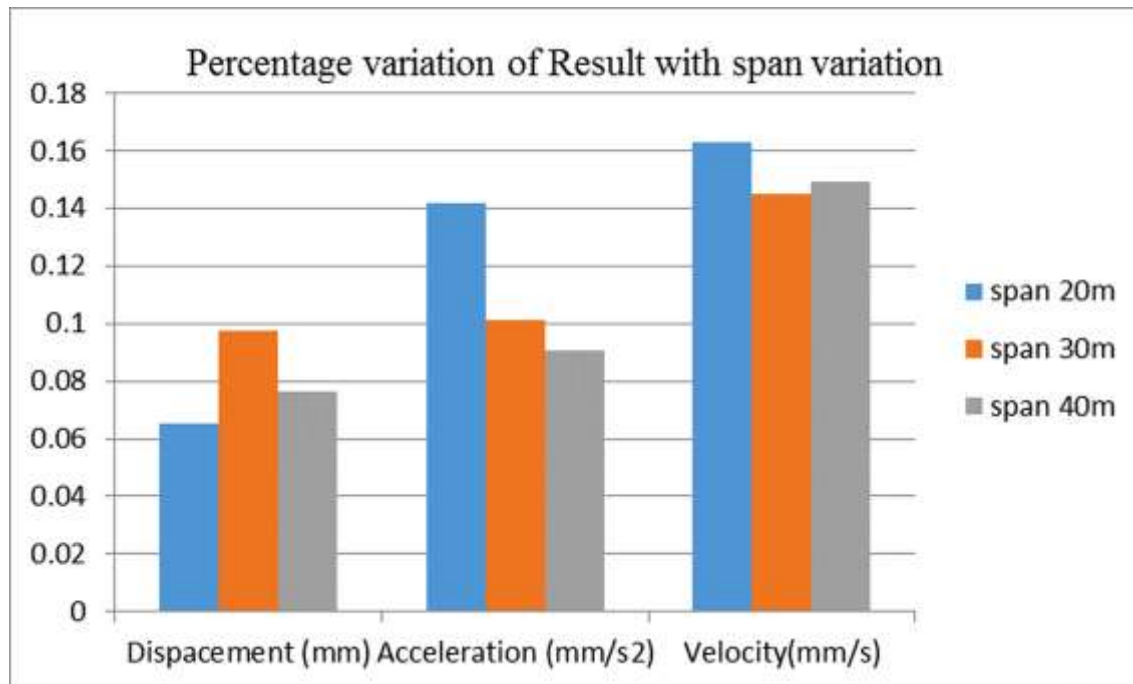


Figure 10. Comparative response trend for 20 m, 30 m, and 40 m arch spans

The span-comparison graph confirms that the 20 m configuration has the lowest response and the 40 m configuration has the highest overall demand. The graphical trend is consistent with the table: increasing span reduces effective stiffness and increases dynamic deformation. Velocity shows the strongest proportional growth, while acceleration changes less sharply. Because the source graph uses its own plotting scale, engineering interpretation is based on the explicit values in Table 9 rather than visual bar height alone.

6. Conclusion

This study assessed the seismic behaviour of straight and arch bridge configurations under the Bhuj 2001 ground-motion record and established a validated analytical basis for later topography-sensitive modelling. Three-dimensional STAAD.Pro models were prepared using consistent concrete properties, bridge geometry, support conditions, dead load, IRC Class AA moving load, and time-history input. The numerical idealisation was checked against manual bending-moment and shear-force calculations for an 80 m validation sub-model; the inner-girder moment differed by only 27 kN-m, approximately 0.80%, between the manual value of 3360 kN-m and the STAAD.Pro value of 3387 kN-m. For the principal 168 m comparison, the arch bridge reduced maximum displacement from 21.8 to 13.8 mm, acceleration from 7.68 to 4.61 mm/s², velocity from 14.72 to 8.83 mm/s, and maximum shear from 9.92 to 5.952 kN. These changes represent reductions of approximately 36.70%, 39.97%, 40.01%, and 40.00%, respectively, making the 168 m arch configuration the better-performing structural form under the reported conditions. Span sensitivity was pronounced: arch displacement increased from 13.1 mm at 20 m to 29.3 mm at 30 m and 30.5 mm at 40 m, while acceleration and velocity also increased. The objectives concerning model development, validation, dynamic comparison, and governing response trends were achieved. The results confirm that arch action limits global seismic response, demonstrate the flexibility penalty of longer spans, and provide a practical framework for evaluating valley geometry, unequal supports, foundation stiffness, and spatially varying excitation in future bridge studies.

7. Future Scope

Future work should apply the validated 168 m arch model to flat, shallow symmetrical, deep symmetrical, and asymmetrical valley profiles while maintaining identical structural properties and earthquake intensity. Site-specific support elevations, unequal pier heights, foundation springs, and soil-structure interaction should be introduced using

verified geotechnical parameters. Multi-support motions should incorporate wave-passage delay, coherency loss, and local amplification rather than arbitrary independent records. Nonlinear material models, bearing and joint behaviour, geometric nonlinearity, and component damage limits should be added where reliable data are available. Analyses using several spectrum-compatible near- and far-field records, uncertainty assessment, optimisation of mitigation devices, and field instrumentation would improve robustness and enable long-term performance and fragility evaluation.

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