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Wind And Seismic Study Of Bridges

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ABSTRACT

There has historically been a nearly global propensity to enhance the capacity demand of the structure in order to counterbalance such events, which occur after every significant earthquake. Only in the last ten years have innovative approaches to solving this issue successfully been created. The emphasis is now placed on serviceability and safety under various earthquake magnitudes in the current international practice of performance-based engineering design. Additionally, it is becoming increasingly clear that in addition to methods for increasing ductility, the toolkit of a structural engineer should also contain devices that dissipate and share energy as well as those that can regulate a system's response. In order to prevent "dislodgement" or "unseating" of the superstructure in the event of strong ground shaking, new methods and systems have been developed. The focus of this work is on how these concepts might be applied to bridge designs that are reasonably earthquake resistant. One of the structural processes that significantly affects bridge design is wind load. The wind load is dynamic by nature. This indicates that it has a time- and space-dependent magnitude. As a result, analysing and modelling such a load and its respective effects on a building may be rather difficult and require a deep understanding of both structural analysis and mathematics.

Keyword: Wind load analysis, Bridge design, Seismic retrofitting, Seismic vulnerability assessment, Seismic isolation systems.

1. INTRODUCTION

Bridges are essential constructions. They are a crucial component of surface transportation networks, and if they fail after a seismic event, relief and restoration efforts will be significantly hampered. Bridges are particularly susceptible to damage and even collapse when subjected to earthquake motions because of their simple structural design. In cases of very strong ground motion, the general design concept for structures is to prevent full collapse. On the methods for evaluating multi-story buildings for seismic risk, there is a wealth of material accessible. Although bridges are an extremely significant structure in any country, there is little effort accessible in the literature for seismic evaluation of existing bridges. Existing bridges receive far less attention. However, bridges are crucial parts of any nation's transportation system. There is currently no seismic design provision in the Indian bridge design codes. Many bridges are built and designed without taking seismic pressures into account. Therefore, it is crucial to assess the capacity of current bridges in relation to the demand for seismic force.

By attempting to lower the seismic forces to levels at or below the structural member's elastic limit, a technique known as seismic isolation can lower inelastic deformations. The essential idea of structural vibration is to be lowered to a frequency that is less than the main energy-containing frequencies of the earthquake. A seismic energy transmission to the system is dissipated by an energy dissipation mechanism provided by an isolation system, which serves another purpose. By isolating the bridge deck from the bridge substructure during earthquakes, which alone is responsible for the majority of the pier base shear, the isolation device, which replaces the conventional bridge bearings, significantly reduces the deck acceleration and, as a result, the forces transmitted to the piers.

1.1 INDIAN CODE RECOMMENDATIONS ON SEISMICITY

For the design of earthquake-resistant bridges, provisions from IS 1893 (Part 1):2002, IS 13920-1993, IRC:6-2016, and IRC:112-2011 can be used. Although there are three mutually perpendicular ways in which an earthquake's ground motion might be resolved, the horizontal direction is the most common. Two thirds of the design horizontal seismic coefficient stated above is assumed to be the design vertical seismic coefficient. Each bridge's component's seismic force is calculated by multiplying its mass by the seismic coefficient's horizontal and vertical components. Although the horizontal force could come from either direction, the vertical force is taken into account independently for each of the two perpendicular horizontal forces. For Zones II and III, only bridges with overall lengths greater than 60 metres or spans greater than 15 metres are required to take seismic forces into consideration during the design process, but all bridges in Zones IV and V must be seismically inspected. In any seismic zone, culverts and small bridges under 10 metres in span are not required to be designed for seismic pressures.

Structures will be subjected to a greater force than the design force level during the expected maximum intensity of earthquake in the various seismic zones. It depends on the structure's plastic range capacity to absorb the kinetic energy from such an earthquake. However, its capacity has not been measured or examined. According to IRC:112-2011 and IS:13920-1993, ductile detailing is required for bridges that are located in zones III, IV, and V of the seismic zone map of IRC:6-2010. The superstructure of a bridge is intended to be an elastic system, and only substructure components that are above the ground are allowed to form plastic hinges. By including confinement reinforcement, concrete's compression zone's ductile behaviour is protected. According to the code, confinement should go at least as far as the point where the compressive strain reaches 0.5 cu2 in length.

IMPORTANCE OF DUCTILITY AND ENERGY DISSIPATION CAPACITY

Better seismic performance requires both ductility and energy dissipation. A structure's ductility, not strength, is what determines how much it can deform. As demonstrated in Fig. 3.1, it functions as a shock absorber, lowering the transmitted force to a manageable level, allowing one to design the members with less strength than the ground motions' requirement for elastic strength. Ductility is defined as the relationship between the maximum displacement beyond the yield limit and the maximum displacement at the yield limit. This must be used in seismic design to prevent using structural sizes that are not financially viable. The degree of redundancy, axial force, steel ratio, structural geometry, etc., are all factors that affect ductility. For the structure to be safe, the ductility demand needs to be significantly lower than the ductility capacity.

There is a common philosophical worry that because the restoring force has been decreased, the displacement experienced by the ductile structure will be greater than that of an elastic structure of equivalent size. But the decreased acceleration offsets this. We may therefore use the equal displacement hypothesis in this situation, allowing us to examine the elastic model and immediately use the results in the development of a ductile construction. This encourages the use of the Performance Based Seismic Design (PBSD) method known as Displacement-Based Design for ductile structures [4]. For bridges, the deck often remains elastic during seismic activity whereas the piers are typically built ductile.

The primary defence against using ductility as a gauge of seismic damage is that seismic activity is cyclical. Therefore, a member must have both deformation capacity and energy dissipation qualities in order to resist seismic force [5]. Energy dissipation capability depends on material properties (grade of concrete and steel), reinforcing details, etc. for a correctly designed and manufactured part.

PROVISIONS FOR IMPROVING DUCTILITY IN BRIDGES

Bridge structures should ideally be built so that the ductile behaviour of the individual elements will distribute the seismic energy and prevent brittle shear failures. This is not always achievable, though, because some bridge design elements may exhibit non-ductile behaviour. A bridge's different components may have varying amounts of ductility, so narrowing down the elastic response spectrum for design purposes may be misleading and lead to some parts being underdesigned. Therefore, the entire structural response should be predicted using the elastic design response spectrum, and the appropriate energy absorption should then be designed into the ductile components. Predicting how a bridge would function in practise during an earthquake is crucial to design. Additionally, ductility must be classified as either being necessary or being available. Another distinction that needs to be made is between the ductility of a structure's overall construction and the ductility of a component's section. The available ductility and the necessary ductility for a specific R employed in the design must match in order to complete the seismic design. Additionally, damping in the structure is explained by ductility. A structure that experiences cyclic loads would have higher damping if there was a considerable energy loss during plastic deformation.

Concrete is recognised to be a brittle material, meaning that when loaded, it breaks down abruptly. But when restrained by reinforcing, concrete can be made ductile. Concrete's ductility is greatly increased by confinement, which also increases the material's strength. By including stirrups, as seen in Fig. 4.1, concrete can be contained. If the stirrups are not attached at 1350 into the core of the concrete, they could open out under the power of an earthquake and prevent the confining effect from working. Furthermore, RC components are only sufficiently ductile in bending action, not in axial or shear action, even with confinement. In order to prevent RC elements from yielding in axial or shear action as well, we must make sure that they do so solely in flexure. This can be made certain by constructing the RC members so that their ability to withstand shear and axial loads is greater than their ability to withstand flexure. 'Capacity Design' is the term used to describe this. A structure can be made to act a certain manner by choosing its flexure, shear, and bending capacities appropriately. A pier may be purposefully constructed to yield at the point where the pile top and pier meet in order to prevent excessive shear from being generated that could harm the foundation or result in collapse. Such intentional sites are referred to be hinges because they allow structural members to move around freely without losing their structural integrity. The potential placement of these plastic hinges in bridge piers is depicted in Fig. 4.2. To make the concrete ductile, such hinge positions should be specifically planned with more stirrups.

I. LITERATURE SURVEY

Applicability

All bridges that are not exempted below in categories (a) and (b) and are supported by piers, pier bents, and arches directly or through bearings must be constructed for the horizontal and vertical forces specified in the following clauses.

There is no need to test for seismic effects on the following bridge types:

Culverts and small bridges with a span of up to 10 metres in all seismic zones

b.) Bridges in seismic zones II and III that meet the requirements for total length under 60 metres and spans under 15 metres.

Seismic Zones

For the purpose of determining the seismic forces, the Country is classified into four zones as shown in Fig. 1. For each Zone a factor 'Z' is associated, the value of which is given in Table 1.

Table 1: Zone factor (Z)

Zone	Zone Factor
v	0.36
iv	0.24
iii	0.16
ii	0.10

Horizontal Seismic Force

The horizontal seismic forces acting at the centers of mass, which are to be resisted by the structure as a whole, shall be computed as follows:

Feq=Ah (Dead Load + Appropriate Live Load)

Feq = Seismic force to be resisted

Ah = Horizontal seismic coefficient = (Z/2) x (l) x (Sa/g)

Z = Zone factor as given in Table 1

I = Importance factor (see Clause 219.5.1.1)

T = Fundamental period of the bridge

In the absence of a careful calculation, the fundamental time period of the bridge member must be determined using any reasonable method of analysis, adopting the Modulus of Elasticity of Concrete (Ecm) as per IRC:112, and taking into account the moment of inertia of the cracked section, which can be taken as 0.75 times the moment of inertia of the gross uncracked section. The approach described in Annex D can also be used to determine the fundamental period of vibration.

Based on the following formulae, Sa/g = Average Responses Acceleration Coefficient for 5 percent dampening of load resisting Elements dependent upon the Fundamental Period of Vibration T

For rocky or hard soil sites, Type I soil with N > 30

- $Sa/g = 1 + 15\,T, \quad 0.00 \leq T \leq 0.10$

  $2.50 \quad , \quad 0.10 \leq T \leq 0.40$

  $1.00/ \;, \quad 0.40 \leq T \leq 4.00$

- For medium soil sites, Type II soil with 10 < N ≤30
- $Sa/g = 1 + 15\,T, \quad 0.00 \leq T \leq 0.10$

2.50 , $0.10 \leq T \leq 0.55$

1.36/ , $0.55 \leq T \leq 4.00$

- For soft soil sites, Type III soil with N < 10
- $Sa/g = 1 + 15\,T$, $0.00 \leq T \leq 0.10$
- 2.50 , $0.10 \leq T \leq 0.67$

Type I - Rock of Hard Soil: Well-graded gravel and sand gravel combinations with or without clay binder, as well as clayey sands that aren't as well-graded or sand clay mixtures (GB, CW, SB, SW, and SC) with N values higher than 30, where N is the standard penetration value.

Type II Medium Soils: All soils with N between 10 and 30, as well as sands or gravelly sands that are poorly graded and have little to no particles. SP when N > 15

Soft Soils of Type III: Every soil besides SP with N10

The allowed bearing pressure shall be established in line with IS 6403 or IS 1883 and the value N (Corrected Value) is at the founding level.

Seismic importance factor (I)

Bridges are built to withstand seismic occurrences that cause damage or failure, as well as other forces of a higher or lower magnitude, depending on the effects of their partial or total unavailability. By multiplying (Z/2) by factor 'I,' which stands for the seismic relevance of the structure, one may get the level of design force. Combination of elements taken into account while determining the consequences of failure and the selection of factor "I" include, among other things:

1. The degree of traffic disruption and the potential for temporary diversion
2. The existence of alternate paths,
3. The time and cost of repairs, which are determined by the severity of the damages, either slight or substantial,
4. Replacement cost and time required for reconstruction in the event of failure,
5. Direct financial harm as a result of its partial or complete non-availability, Table 19 lists critical factors for various bridge types.

Importance Factor

Live load components

6. When acting perpendicular to the direction of traffic, the seismic force caused by the live load must be taken into account rather than when acting in that direction.

7. Twenty percent of the live load (excluding impact factor) must be used to determine the horizontal seismic force in the direction perpendicular to the traffic.Calculate the vertical 8. seismic force using 20% of the live load (without the impact factor).

Water current and depth of scour

For design purposes, the depth of scour under seismic conditions must be 0.9 times the maximum scour depth. The average of the yearly maximum design floods should be used to determine the flood level for calculating hydrodynamic force and water current force. For river bridges, the average may ideally be based on data collected over a period of seven years, or, in the absence of such data, on local inquiries.

WIND LOAD

This provision applies to bridges with a height of pier up to 100 m or with typical spans with individual span lengths up to 150 m. Specialist literature must be utilised to calculate the design wind load for all other bridges, including cable-stayed, suspension, and ribbon bridges.

The terrain of the surrounding area, the fetch of the terrain upwind of the site location, the local topography, the height of the bridge above the ground, the horizontal dimensions, and the cross-section of the bridge or its component under consideration all affect the wind pressure acting on a bridge. Gusts are to blame for the highest pressure because they produce temporary and localised variations in the mean wind pressure.

The wind speed at the bridge's location must be determined using the basic wind speed map described in. Based on hourly mean wind speed and pressure, wind force intensity will be determined. For bridges located in flat, unobstructed terrain and plains, the hourly mean wind speed and pressure values in the table correspond to a basic wind speed of 33 m/s with a return time of 100 years. In accordance with the location of the bridge and other fundamental wind conditions, the hourly mean wind pressure must be updated as necessary.

Hourly Mean Wind Speed and Wind pressure

	Bridge Situated in			
	Plain Terrain		Terrain with Obstructions	
H (m)	V_z (m/s)	P_z (N/m^2)	V_z (m/s)	P_z (N/m^2)
Upto 10 m	27.8	463.7	17.8	190.5
15	29.2	512.5	19.6	230.5
20	30.3	550.6	21	265.3
30	31.4	590.2	22.8	312.2
50	33.1	659.2	24.9	373.4
60	33.6	676.3	25.6	392.9
70	34	693.6	26.2	412.8
80	34.4	711.2	26.9	433.3
90	34.9	729	27.5	454.2
100	35.3	747	28.2	475.6

H = the average height in metres of exposed surface above the mean

Vz= Hourly mean speed of wind in m/s at height H.

Pz= Horizontal wind pressure in N/m2 at height H

Design Wind Force on Superstructure

The superstructure must be built to withstand simultaneous vertical loads and horizontal forces caused by wind that act in both transverse and longitudinal directions. When a structure is straight or curved in plan, the assumed wind direction must be perpendicular to either the longitudinal axis or the axis selected to maximise wind-induced effects.

The transverse wind force on a bridge superstructure shall be estimated as specified in Clause 209.3.3 and acting on the area calculated as follows:

For a deck structure:

a) The area of the structure as seen from an elevation, including the floor system and railing, shall be taken into consideration, minus the area of perforations in the hand railing or parapet walls. For crash barriers, railings, and open and solid parapets, the solid area of the element's usual projected elevation must be taken into account.

The transverse wind force FT (in N) shall be taken as acting at the centroids of the appropriate areas and horizontally and shall be estimated from:

$$FT = PZ \times A1 \times G \times CD$$

where, PZ is the hourly mean wind pressure in N/m2, A1 is the solid area in m2 G is the gust factor and CD is the drag coefficient depending on the geometric shape of bridge deck.

For highway bridges up to a span of 150 m, which are generally not sensitive to dynamic action of wind, gust factor shall be taken as 2.0.

The drag coefficient for slab bridges with width to depth ratio of cross-section, i.e $b/d \geq 10$ shall be taken as 1.1.

In the case of bridge decks supported by a single beam or box girder, CD should be calculated as 1.5 for a b/d ratio of 2, and 1.3 for a b/d of less than 6. Interpolation of CD is required for intermediate b/d ratios. When a deck is supported by two or more box girders or beams, and the ratio of the clear distance between the boxes to the depth is not greater than 7, the combined clear distance (CD) for the structure is 1.5 times the CD for the single beam or box.

When the deck is supported by two or more plate girders, the combined structure's CD must not exceed 2(1+c/20d), where c is the center-to-centre distance between adjacent girders and d is the depth of the windward girder. For a deck supported by a single plate girder, the value must be taken as 2.2.

For beam/box/plate girder bridges and truss girder bridges, the longitudinal force on bridge superstructure FL (in N) shall be taken as 25% and 50%, respectively, of the transverse wind load as computed as per Clause 209.3.3.

An upward or downward vertical wind load FV (in N) acting at the centroid of the appropriate area, for all superstructures shall be derived from:

$$FV = PZ \times A3 \times G \times CL$$

Pz= Hourly mean wind speed in N/m2 at height H

A3== Area in plain in m2

CL= Lift coefficient which shall be taken as 0.75 for normal type of slab, box, I-girder and plate girder bridges. For other type of deck cross-sections CL shall be ascertained either from wind tunnel tests or, if available, for similar type of structure. Specialist literature shall be referred to.

G= Gust factor

The expression must be used to calculate the transverse wind load per unit exposed frontal area of the live load, but CD against must be considered as 1.2. The full length of the superstructure viewed in elevation in the direction of the wind as stated in clause, or any portion of that length causing critical reaction, multiplied by a height of 3.0 m above the road way surface, is what is referred to as the exposed frontal area of live load. Neglected areas are those that are below the top of a solid barrier.

The transverse wind load as estimated above will be considered as the longitudinal wind load on live load. Both loads must be applied at once, acting at a height of 1.5 metres above the road.

When the wind speed at deck level surpasses 36 m/s, the bridges are not deemed to be carrying any live load.

In the case of a cantilever construction, in addition to the calculated transverse wind effect, an upward wind pressure of PZ x CL x G N/m2 on the bottom soffit area must be assumed on the stabilising cantilever arm. In addition to the aforementioned loads, other defined loads must also be taken into account.

Design Wind Forces on Substructure

The substructure must be built to withstand both wind loads occurring directly on it and wind loads communicated to it from the superstructure. It is important to take into account loads for both normal and skewed wind directions relative to the superstructure's longitudinal centerline.

For the purpose of computing FT, A1 will be used as the solid area in the usual projected elevation of each pier. Shielding shall not be taken into account.

Conclusion

When a bridge is being built in an earthquake or seismic zone, earthquake loads must be taken into account. During an earthquake, they produce both vertical and horizontal stresses. The self-weight of the structure has a major influence on the amount of forces applied. Greater forces will be applied if the building is heavier.Another crucial element in the design of the bridge is the wind load. Wind load may not be significant for bridges with small spans. However, when designing the substructure of medium span bridges, wind load should be taken into account. Wind load is taken into account when designing large-span bridges.

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