A DISSERTATION ON

BEHAVIOUR OF PIERS IN CURVED RAILWAY BRIDGES UNDER MOVING LOAD

(Major Project – II)

Submitted in Partial Fulfilment of the Requirements
for the Award of the Degree of

MASTER OF TECHNOLOGY
(STRUCTURAL ENGINEERING)

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CERTIFICATE

This is to certify that the major project-II entitled "**BEHAVIOUR OF PIERS IN CURVED RAILWAY BRIDGES UNDER MOVING LOAD**" being submitted by me is a bonafide record of my own work carried by me under the guidance of **Dr. Nirendra Dev, Professor** in partial fulfilment of the requirements for the award of the Master of Technology in Civil Engineering with specialization in **STRUCTURAL ENGINEERING, DELHI TECHNOLOGICAL UNIVERSITY** (Formerly Delhi College of Engineering) DELHI-110042.

The matter embodied in this project has not been submitted for the award of any other degree.

EKTA GARG
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CERTIFICATE

This is to certify that the above statement made by the candidate is correct to the best of my knowledge.

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ABSTRACT

Historically railway bridges have been built only in straight providing curves on approach embankments. Wherever, curved bridges have been provided, curvature was too flat to produce any significant centrifugal force. The codes have accordingly provided for considering the centrifugal force as static force (=mv^2/r).

However, with the introduction of Metro Services which have to pass thro congested city areas, bridges with sharp curves have to be provided with high piers which is a significantly changed scenario than conventional railway bridges, however, no separate guidelines are available on the subject.

Since the load from train is transferred to bridges substructure through rail wheel contact as moving point load instead of line load, the resulting action is a time variable load on substructure and hence its effect is more close to dynamic load than a static load. From the study carried out in Major Project-I, it has been noted that when centrifugal load is applied as a dynamic load instead of static load, the dynamic amplification in pier deflection and resulting internal forces depends upon train speed. Tendency of resonance has also been noted at a critical speed.

The proposed study aims at studying the behaviour of piers in curved bridges by varying various parameters, viz., i) span length, ii) Weight of span, iii) Height of pier, iv) Size (Dia) of Pier and v) Speed of train by way of dynamic analysis. The results obtained will be compared with the results obtained from static method as per existing codal provisions. The combinations of span, pier height and train speed resulting in resonance or high amplification factor will be identified and recommended to be avoided for field application. The combinations not leading to resonance or high dynamic amplification factor within operational speed of trains will also be identified for safe use.

CHAPTER – 1

INTRODUCTION

1.1 PREAMBLE:

Bridges are important component of any land transport system. In earlier days, bridges were built only in straight & level and any deviation in alignment was achieved by providing curves on approach embankments[10]. This has avoided the need to consider the effect of loads moving in curve. With the spread of civilization and advancement of technology, the need was felt to provide bridges in curve, specially in hilly areas, however, the curvature was kept too flat to produce any significant centrifugal force and forces due to curvature were considered as static forces using conventional concept of centrifugal force.

Further, with the introduction of Metro Services which have to pass through congested city areas over already existing flyovers/interchanges, bridges with sharp curves have to be provided with high piers which is a significantly changed scenario than conventional bridges, especially considering the high speed of metro trains.

Since the load from train is transferred to bridge substructure through rail wheel contact as a train of moving point loads instead of line load, the resulting action is a time dependent load on substructure and hence its effect is more like a dynamic load than a static load. Accordingly, the centrifugal load needs to be applied as a dynamic load at the centre of gravity of moving train instead of static load (as being done at present).

From the study carried out in Major Project-I, it has been noted that when centrifugal load is applied as a dynamic load instead of static load, significant dynamic amplification takes place. Hence, there is need to further study the influence of dynamic effects of moving train load on curved bridges of various spans.

1.2 ELEMENTS OF BRIDGES

A typical bridge essentially has following elements:

1. Horizontal (plan) alignment
2. Vertical (elevation) alignment
3. Abutment
4. Pier
5. Superstructure
6. Bearings

1.2.1 Horizontal (plan) alignment: Horizontal alignment of the bridge defines the geometry of a bridge in plan. Normally bridges are in straight, however, in case of horizontally curved bridges, radius of curve, deflection angle, location of tangent points of curve and length of transition curve need to be defined.

1.2.2 Vertical (elevation) alignment: Vertical alignment of the bridge defines the geometry of a bridge in elevation. Normally bridges are in level, however, in case of vertically curved bridges, grade change point, gradient on each side of change point and radius of vertical curve need to be defined. Normally, vertical curves are provided in the form of parabola and hence no transition curve is required.

1.2.3 Abutment: These are provided on each end of the bridge to retain the earth on approaches besides supporting the last span.

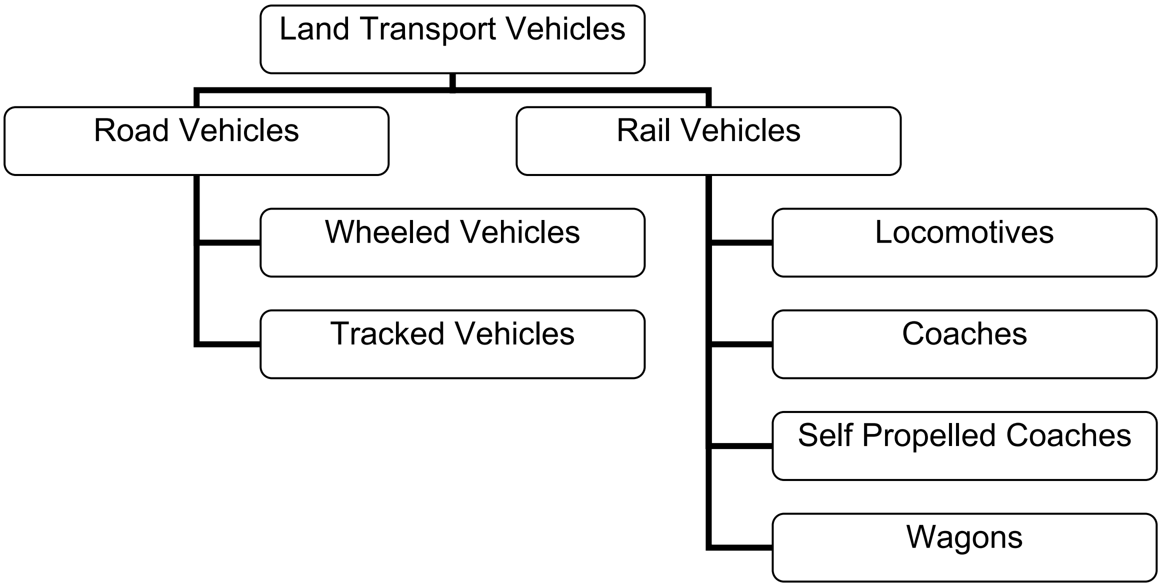
1.2.4 Pier: These are provided between abutments to support the two spans on either side of pier.

1.2.5 Superstructure: This is the part of the bridge on which road/track is laid. Usually, it is in the form of slabs or girders with deck. It spans from abutment to next pier or abutment.

1.2.6 Bearings: These are provided between superstructure and abutment/pier to allow relative thermal and other movement between superstructure and abutment/pier.

1.3 LOADING DUE TO VEHICULAR TRAFFIC

Land transport vehicles can broadly be classified into following categories.



Whereas road traffic consists of a mix of different type of vehicles (Light/Heavy, Small/Large, Wheeled/Tracked, densely/distantly placed) for the given duration of time, rail traffic is more or less uniform as usually a train consists of a number of similar type of vehicles (coaches/wagons) with or without locomotive. Loads to be considered for design of road and rail bridges have been specified in respective codes (IRC:6[7] for road bridges and Bridge Rule[11] for rail bridges in India).

In case of bridges when loading on one span or pier is being considered, the pattern of loading has distinct effect on dynamic properties of loading. Road traffic is random in nature with a mix of vehicles with varying spacing resulting in noise type of loading in time. However, rail traffic consists of a train of same/similar vehicles placed at uniform spacing thus resulting a periodic loading in time. In case of self propelled trains, e.g., EMU (Electrical multiple units), DMU (Diesel multiple units), Metro trains etc. having a number of identical vehicles, the cyclicity of loading in terms of axle load and spacing further becomes more uniform thus resulting in a pronounced periodic dynamic component of loading. The effect of this dynamic component is considered by using an impact factor resulting in enhanced vertical loads on the bridges.

In case of curved bridges, a centrifugal force acts on the vehicles moving in a curved path. This force is transferred to the bridge through road-wheel or rail-wheel contact. This force ultimately acts on the bridge sub-structure (abutment/piers) which needs to be designed for the same.

1.4 WORK CARRIED OUT IN MAJOR PROJECT-I

To understand the response of piers in curved bridges, an study was undertaken in Major Project-I, in which, dynamic analysis of an 8 m high pier supporting curved 28m span of 360 m radius on either side was carried out for the centrifugal force applied by train moving at different speeds. The calculated dynamic lateral load on pier due to centrifugal force was compared with that obtained using present codal provisions considering centrifugal force to be a static force.

It was observed that dynamic effects of centrifugal force on pier are higher than that obtained from codal provisions which are based on static analysis. Maximum amplification factor of 1.23 was observed for the pier analysed in the study carried out in Major Project-1. Tendency of resonance was also noted at a speed of 20.5 m/s. i.e., 73.8 km/h, which is within the operational speed of metro trains.

1.5 OBJECTIVE OF THE STUDY

The proposed work aims at studying the behaviour of piers in curved bridges in dynamic analysis by varying various parameters, viz., i) length of span, ii) weight of span, iii) height of pier, iv) dia of pier and v) train speed. The results obtained will be compared with the results obtained from static method as per existing codal provisions. The combinations of span, pier height and train speed resulting in resonance or high amplification factor will be identified. The combinations not leading to resonance or high dynamic amplification factor within operational speed of trains will also be identified.

On the basis of study, the span-pier height combinations indicating tendency for resonance within operational speed of metro trains will be recommended to be avoided for field use. Recommendations for use will also be made for span-pier height combinations which are found safe (resulting in negligible/ low dynamic amplification) within operational speed of metro trains.

1.6 SCOPE OF THE STUDY

The scope of study will include and be limited to calculation of lateral dynamic forces developed in piers of different heights of curved metro train bridge of different spans due to centrifugal force acting on train load moving at different speeds using dynamic analysis of piers. The results will be compared with centrifugal force calculated as static force in accordance with present codal provisions and Dynamic Amplification Factor (DAF) will be calculated for each case.

Based on DAF obtained, the spans will be categorised as critical (high DAF) and non-critical (low DAF) from dynamic analysis point of view. Spans leading to resonance under passage of trains within operation speed of trains will also be identified.

CHAPTER-2

LITERATURE REVIEW

2.1 HISTORIC DEVELOPMENT OF BRIDGES:

Since primitive small bridge of ancients to bewildering modern bridges of large spans, a bridge has always offered man the satisfaction of successfully crossing an obstruction. Ram Setu joining Pamban island and Mannar island, which is more than 4000 years old, is the oldest known remains of a bridge form[10]. The oldest existing bridge consisting of 20 pointed arches of 7.5 m span and dating back to about 350 BC is at Khorsbad in Babylonia[10].

However, significant development in bridge engineering started taking place in 18 century with beginning of transition from timber/stone to steel bridges. Another mile stone in bridge history was use of reinforced concrete which started in early 19th century and used for large scale rebuilding of damaged bridges after world war-I. In 1928, Freyssinet developed the concept of pre-stressing of concrete[10] and obtained patient for it. The technology was subsequently used for massive rebuilding of bridges in Europe after world war-II. Thereafter significant developments have taken place in bridges, both in material and computational techniques, leading to present day modern bridge engineering resulting in construction of long span cable stayed and suspension bridges creating the engineering marvels of modern age.

In the past, engineers tried to place bridges perpendicular to the crossing, i.e., rivers, gorge etc. and adjusted the alignment of road at approaches to suit the site conditions. This made mathematical analysis of bridges simpler. This practice is still followed, where ever possible, even after availability of modern computational tools.

However, in situations where this simplification is not considered practical, e.g., in densely populated area, bridge interchanges, hilly terrain etc, the bridge shape in plan has to be curved to suit the alignment. In such cases, more powerful analytical techniques have to be resorted to take into account the effect of curve.

2.2 A CRITICAL APPRAISAL OF FORCES ON BRIDGES

Depending upon the type (rail/road, straight/curved), bridges are subjected to a number of forces which have been given in detail by IRC:6 for road bridges[7] and *Bridge Rule* for railway bridges[11]. These includes

1. Dead Load
2. Super-Imposed Dead Load
3. Vehicular Live Load
4. Centrifugal Forces
5. Raking Forces
6. Footpath Live Load
7. Wind Load
8. Seismic Load
9. Thermal Load
10. Hydraulic Forces

The details of these forces have been discussed in following text.

2.2.1 DEAD LOAD

Dead load is the weight of the structure itself together with the permanent loads carried thereon. These include, weight of superstructure, footpaths, kerbs, protection barriers/railings etc. Dead load is calculated using volume of material as per drawing and density of material as given in relevant code.

2.2.2 SUPER IMPOSED DEAD LOAD

Super imposed dead load is the weight of the temporary and semi-permanent fixtures/structures placed on the bridge. In case of road bridges, this includes weight of road surface, services etc. In railway bridges, this includes weight of track with ballast, wearing coat, services etc.

2.2.3 VEHICULAR LIVE LOAD

IRC:6 specifies the weight and configuration of road vehicles to be used for design of bridges. The vehicle definition broadly includes the no of axles, axle spacing, axle load for each axle and with of axle. The road vehicles can broadly be classified in two categories, viz, wheeled and tracked vehicles. Wheeled vehicles represents the trucks, trailers, lories etc with pneumatic tyres. Tracked vehicles represents the vehicles with crawler/track, e.g., cranes, dozers, army tanks etc. Presently 70R and Class A loading are being used for design of bridges.

Bridge Rules specifies the vehicular live load to be used for railway bridges. The load definition broadly includes axle load & axle spacing for locomotives and a uniformly distributed load for trailing load for each type of gauge of track. As there are different type of rolling stocks having different loading characteristics are being used in railway, a standard design load has been specified covering all commonly plying vehicles. As the length of trains is much more than length of a span, no of axles are not specified and full span is supposed to be occupied by live load.

For metro trains, there is no separate standard and load for passenger coaches @ 65 T per coach is being used in design. The detailed vehicle loads for metro coaches are available on web sites of metro coach manufacturers and will be discussed in detail in other chapter.

2.2.4 CENTRIFUGAL FORCES

When an object moves in a curved path, its direction of motion changes continuously for which a centrifugal force is required to be applied on the object in a direction normal to the direction of motion. In case of rail/road vehicles, part of this force is generated by providing super elevation/cross slope to the track/road and rest is generated as friction between wheel and rail/road.

For road vehicles, Cl. 212 of IRC:6-2010 specifies that all portions of a curved bridge should be designed for the action of centrifugal force[7]. For railways, Cl. 2.5.3 of Bridge Rule[11] specifies the centrifugal force is considered in design of bridges. The provisions will be discussed in detail subsequently.

2.2.5 RACKING FORCES

Due to coning of railway wheels, motion of vehicles is not in a straight line but it also has small sinusoidal movement due to which a lateral force is also applied on track. Cl. 2.9.1 of bridge rule[11] specifies that racking forces @ 600 kg/m shall be applied as moving load.

2.2.6 FOOTPATH LIVE LOAD

Wherever footpaths are provided on bridges, a footpath live load is also to be considered along with vehicular traffic. Cl. 206.1 of IRC:6[7] specifies that it should be considered as 400 kg/m^2 for normal conditions, however, if over crowding is expected over the bridge due to proximity to town, pilgrimage or fairs areas, it should be considered as 500 kg/m^2. This intensity of live load is to be modified for longer spans as per provisions given in the code.

Cl. 2.3.2.1 of Bridge Rule[11] specifies that footpath live load should be considered as 415 kg/m^2 when crowding is not expected and 490 kg/m^2 when crowding is expected.

2.2.7 WIND LOAD

Wind Forces act both on bridge and vehicular traffic. Permitted stresses in bridges are also allowed to be increased when wind load is considered. For limit state design, reduced partial load factors are allowed to be to be used. For road bridges, Cl. 209 of IRC 6[7] specifies in detail the wind load to be considered in design of bridges. The provisions are by and large in line with those given in IS:875 (Pt-3)[3].

For railway bridges, provisions given in Bridge rule[11] are much simpler. For spans upto 20m, wind+racking forces are to be considered as 900 kg/m (Cl. 2.9.2). For larger bridges, wind speed as per local metrological data is to be considered. However, if metrological data is not available, IS:875 (Part-3) -1987[3] may be used to determine the wind pressure. Further, calculation of wind load on structure is based on horizontally projected area of structure and live load.

2.2.8 SEISMIC LOAD

Detailed provisions for Seismic Loads on bridges have been made both in IRC:6 for Road bridges and Bridge Rule for Railway bridges. Permitted stresses in bridges are also allowed to be increased when seismic load is considered. For limit state design, reduced partial load factors are allowed to be to be used. Both the codes dispense with considering the seismic loads for small bridges.

Cl. 219 of IRC:6 specifies the seismic loads to be considered for road bridges[7]. However, bridges upto 10 m span in all zones and upto 15 m span with 60 m overall length in Zone II & III have been exempted from seismic design.

Cl. 2.12 of Bridge Rule specifies the seismic loads to be considered for rail bridges[11]. However, slab, box and pipe culvers in all zones and upto 15 m span with 60 m overall length in Zone II & III have been exempted from seismic design.

2.2.9 THERMAL LOAD

Temperature has two effects on the structure:

a. Change in overall length of the bridge due to increase or decrees in overall (effective) temperature of the bridge causes stresses in bridge due to friction in bearing or due to restraint against movement.
b. Differential change in temperature across section of bridge superstructure causes differential temperature stresses in bridge superstructure.

For road bridges, Cl. 215 of IRC:6 specifies temperature effects to be considered on bridges in detail[7]. Permitted stresses are also allowed to be increased when considering differential temperature effects.

For railway bridges, Cl. 2.6 & 2.7 of Bridge Rule specifies temperature effects to be considered on bridges due to overall change in temperature[11]. However, it is limited to frictional effect only.

2.2.10 HYDRAULIC FORCES

For bridges in water streams, substructure of the bridge experiences forces due to water current. For road bridges, Cl. 210 of IRC:6 specifies[7] the forces due to water current as :

$$P = 52\ KV^2$$

Where

P	= intensity of pressure due to water current (kg/m^2)
V	= velocity of water current (m/s)
K	= a constant depending upon pier geometry (0.45 to 1.5)

For railway bridges, similar provisions have been made in Cl. 5.9 of Bridge Sub-Structures & Foundation Code[12]. The force due to water current has been specified as:

$$P = KAV^2$$

Where

P	= Total pressure due to water current (kg)
A	= Area of part exposed to water current (m^2)
V	= velocity of water current (m/s)
K	= a constant depending upon pier geometry (24 to 79)

2.3 VIBRATIONS DUE TO MOVING TRAINS

2.3.1 Characteristic of Train Load

The train load is transferred to the track as moving rail-wheel contact points resulting in a series of moving vertical and lateral point loads. Accordingly, the loading on bridge due to moving train, when represented as a function of time, also have a significant harmonic component besides static load.

Further, due to coning of railway wheels, motion of vehicles is not in a straight line but it also has small sinusoidal movement due to which a lateral force is also applied on track.

If the frequency of these harmonic components of load is close to the natural frequency of the superstructure/ substructure or ground, there may be a tendency of resonance resulting in large displacement and forces in the system and under such conditions, force calculations from static theory may not be reliable.

2.3.2 Impact due to Moving Train Load

The effect of load of a moving vehicle gets amplified over its static load due to various reasons mentioned above. This is usually termed as impact. The value of impact has been specified by various codes for both road and railway bridges and considered while designing the structures. Though the amplification in vertical direction has been accounted for by various codes by defining the value of impact, its lateral component is unaddressed.

2.3.3 Ground Vibrations caused by Moving Trains

As mentioned in Para 2.3.1 above that forces caused by moving train load has some harmonic component as well which is responsible for its dynamic effects. When frequency of this harmonic component matches with natural frequency of structure, the resonance occurs resulting in significant vibrations in structures. Since historically metro tracks have been made underground in western countries, the vibrations due to underground metro trains have been widely studied by various scholars and researchers.

Xiaojing et al. (2008)[17] has studied the Vibration impacts on adjacent sensitive buildings induced by metro trains. Bahrekazemi (2004)[1] has studied Train-Induced Ground Vibration and prepared a model for its prediction. It has been stated that ground-borne vibration caused by train traffic is influenced by factors as wheel and rail roughness, discrete track supports, dynamic characteristics of the rolling stock, rail support stiffness, railway structure design, soil characteristics, building structure design, train speed and frequency content of the generated ground-borne vibration.

Wenbin et. al. (2012)[14] has carried out a study on vibration bounce area caused by metro based on the pulse impact experiment. He has observed that the ground surface vibrations depend on the main vibration frequency band of the vibration source and the propagation characteristics of the stratum.

Xia et. at. (2001)[15] has studied vibration effects of light rail train-viaduct system on surrounding environment. He has observed that ground vibration near the bridge is mainly induced by dynamic impact of train as moving load and increases with increase in speed of train and becomes significant as speed changes from 60 km/h to 80 km/h.

2.4 DYNAMIC ANALYSIS OF STRUCTURES

2.4.1 Characteristic of Dynamic Problem

Dynamic considerations are often more complex and complicated than its static counterpart. In dynamic problem, magnitude, direction and/or position of a load are varying with time. Similarly, the structural response to a dynamic load is time varying too. Due to this, a dynamic problem instead of having a single solution as a static one does, has a succession of solutions corresponding to all times of interest in the response history[4].

Dynamic problem essentially imply an addition of inertia and damping to the elastic resistance force[2, 4]. In dynamic condition, inertial and damping forces are produced, which resist accelerations of the structure. When a dynamic load is applied to a structure, the resulting response depends not only upon the load, but also upon inertial and damping forces. Therefore, the corresponding internal response of the structure must equilibrate, not only to the externally applied forces, but also to the inertial forces resulting from the accelerations of the structure.

If the inertial forces represent a significant part of the total load, the dynamic character of the problem has to be taken into account in the calculations. If the inertial forces are negligibly small, the analysis of the response may be regarded as static, even though the load and response may be varying with time[4].

2.4.2 Resonance in Railway Bridge

Resonance is a dangerous phenomenon, which may occur in railway bridges under higher speeds due to regularly spaced axle groups of the trains. In case of resonance, due to high accelerations, excessive bridge deck vibrations[2] may cause loss of wheel-rail contact and exceeding the stress limits of the bridge structure. Therefore, dynamic analysis of bridge structures is necessary in case resonance is anticipated under moving train load.

The effects of maximum dynamic load occur at the resonant peaks. Risk of resonance arises when the excitation frequency of the loading, or a multiple of it, coincides with a natural frequency of the bridge structure. As the speed of the train increases, when it passages the bridge, the excitation frequency of the train may approach the natural frequency of a mode of vibration of the bridge[16].

When resonance occurs, the dynamic responses of the structure increase very rapidly. Occurrence of resonance depends on the number of groups of regularly spaced loads, the damping of the structure, and the nature of loading and the characteristics of the structure. Fig. 2.1 presents the response of a structure for various damping and frequency ratios[4]. It is noted that the magnitude of the resonant peaks is highly dependent upon structural damping and lower value of the damping of the structure gives higher resonance peaks. In such situations, traffic safety on the railway bridge may be compromised[6].

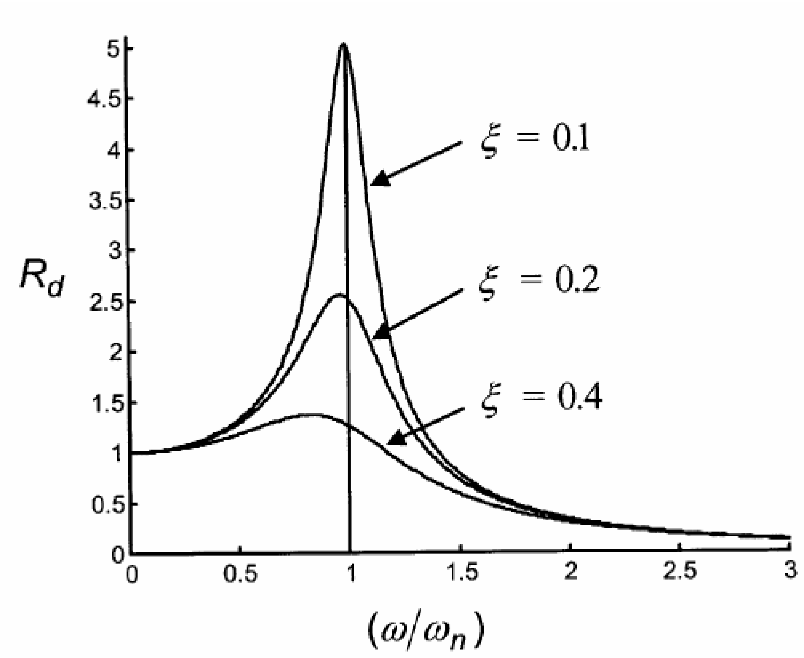


Figure 2.1 Response of structure under dynamic conditions

2.4.3 Dynamic Amplification Factor

Usually, the dynamic loads results in increasing of the bridge response, when compared to static loads. The dynamic response is commonly presented in terms of Dynamic Amplification Factors. These factors indicate that how many times the static response, of a railway bridge due to moving traffic, must be magnified in order to cover the additional dynamic loads. Most frequently, the Dynamic Amplification Factor is defined as a dimensionless ratio of the absolute maximum dynamic response to the absolute maximum static response[2, 4, 6].

$$\text{Dynamic Amplification Factor} = \frac{\text{Absolute Maximum Dynamic response}}{\text{Absolute Maximum Static response}}$$

2.4.4 Damping in Structures

In dynamic analysis, the structural damping is an important key parameter but they are often not well known. The response of a bridge structure due to moving loads depends heavily on the structural damping, especially near resonance.

Damping is essentially dissipation of energy from vibrating structure and is a very complex phenomenon. There are both internal and external sources of damping of bridge structures. The internal sources of damping include viscous internal friction of building materials, non-homogeneous properties and cracks. The external sources of damping includes friction in supports and bearings, friction in the permanent way and in the joints of the structure, viscoelastic properties of soils and rocks below or beyond the bridge piers and abutments[4, 6, 16].

2.5 EQUATION OF DYNAMIC EQUILIBRIUM

A single degree of freedom system can be represented as shown in Fig. 2.2. For a single degree of freedom system, at any point of time, t, equation of dynamic equilibrium can be written as[8]:

$$M\ddot{x} + C\dot{x} + Kx = f \qquad (2.1)$$

Where

M = Mass of the structure
C = Damping in the structure
K = Stiffness of the structure
$\ddot{x}$ = Acceleration of mass
$\dot{x}$ = Velocity of mass
x = Displacement of mass
f = externally applied force at time, t

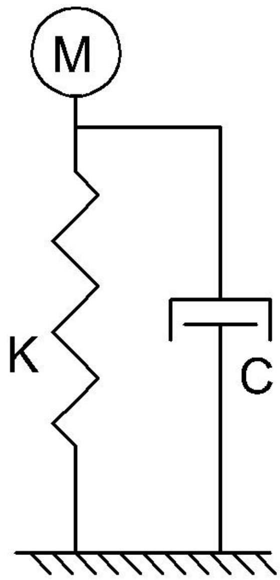


Fig. 2.2 A SDOF System

Whereas, mass, M and stiffness, K can be easily determined from bridge parameters, damping C needs some discussion as very limited information is available on modelling of damping in structures. It is customary to assume that damping is proportional to mass[8], i.e.,

$$C = \alpha M \tag{2.2}$$

Where

α $= 2\xi\omega$

ξ $=$ damping factor

ω $=$ natural frequency of structure

2.6 EXPLICIT TRANSIENT DYNAMIC ANALYSIS SCHEME

For simple excitation function, f, response of the structure can be obtained by solving the differential equation 2.1 in conventional manner. However for complex excitation functions, solution of equation of dynamic equilibrium requires adoption of numerical time stepping procedures or integration schemes for obtaining response of structure in time. Such time integration schemes can be broadly classified as explicit or implicit schemes[8]. Explicit schemes are simpler to implement but conditionally stable and smaller time step is required for stability of solution. Implicit schemes are though unconditionally stable, requires intricate iterative calculations. For linear analysis, explicit schemes works well subject to selection of appropriate time step and hence selected for the present study.

In explicit time integration scheme, velocity and acceleration at any time step are calculated using central difference approximation. If x_{n-1}, x_n and x_{n+1} are displacements at time step n-1, n and n+1 respectively, then at time step n, acceleration can be written as[8]

$$\ddot{x} \approx a_n = \frac{x_{n+1} - 2x_n + x_{n-1}}{\Delta t^2} \tag{2.3}$$

and velocity can be written as

$$\dot{x} \approx v_n = \frac{x_{n+1} - x_{n-1}}{2\Delta t} \tag{2.4}$$

Where, Δt is time step or interval at which sampling of parameters (displacement, force, velocity and acceleration etc.) is being done.

Substituting values of velocity and acceleration from Eqn 2.3 & 2.4 respectively in Eqn 2.1 for time step n,

$$M\left\{\frac{x_{n+1}-2x_n+x_{n-1}}{\Delta t^2}\right\}+C\left\{\frac{x_{n+1}-x_{n-1}}{2\Delta t}\right\}+Kx_n=f_n \tag{2.5}$$

Eqn 2.5 can be rearranged to give

$$x_{n+1}=\left[M+\frac{\Delta t}{2}C\right]^{-1}\left\{\Delta t^2\left(f_n-Kx_n\right)+2Mx_n-\left(M-\frac{\Delta t}{2}C\right)x_{n-1}\right\} \tag{2.6}$$

or

$$x_{n+1}=f\left(x_n,x_{n-1}\right) \tag{2.7}$$

Therefore, it is possible to calculate displacements at time step (n+1) explicitly in terms of displacements at time step n and (n-1), hence the procedure is termed as explicit time integration scheme.

As already mentioned, explicit time integration schemes are conditionally stable and require time step to be small enough. However, from computational economy point, time step should be as large as possible. For stability of central difference scheme, time step should be limited by the expression[8]:

$$\Delta t\leq\frac{2}{\omega_{\max}} \tag{2.8}$$

For SDOF system, $\omega_{max} = \omega_n = 2\pi/T_n$. Hence, Eqn 2.8 can be written as:

$$\Delta t\leq\frac{T_n}{\pi} \tag{2.9}$$

Using Eqn. 2.6 and selecting appropriate time step from Eqn. 2.9, response of the structure having SDOF can be calculated for each time step successively.

CHAPTER - 3

FORMULATION OF PROBLEM

3.1 PREAMBLE

The provisions of centrifugal forces given in Indian codes have been discussed in Chapter-2. It is noted that these provisions follow the classical physics for calculation of centrifugal force. These provisions may work well for road bridges where speeds are relatively low and traffic pattern is by and large random in nature both in terms of weight and spacing of vehicle axles thus avoiding the possibility of resonance.

However, in case of railway bridges, speeds are high and traffic pattern on a given span is repetitive in nature due to axles of almost equal weight placed in a patterned manner. The centrifugal force is also transferred to the bridge as moving load, concentrated at points of rail-wheel contact. The effect of moving load on a structure is more close to dynamic loading then static loading. Amplification of force may also take place under dynamic conditions. Hence, there is need to investigate the effect of centrifugal force due to moving train load on piers of curved bridges using dynamic analysis.

3.2 CONSIDERATION OF CENTRIFUGAL FORCES FOR CURVED BRIDGES - PRESENT PRACTISE

At present, centrifugal force on bridges is considered as static force calculated from classical physics. No factor is applied on this force to take into account the amplification under dynamic effects. From basic physics, the centrifugal force is defined as:

$$C = \frac{mv^2}{r} \tag{3.1}$$

Where

C = centrifugal force acting on the body (N)

m = mass of the body (kg)

v = velocity of the body (m/s)

r = radius of curved path (m)

For road vehicles, Cl. 212 of IRC:6-2010 specifies that all portions of a curved bridge should be designed for the action of centrifugal force acting at a height of 1.2 m above road level. The value of force should be determined as[7]:

$$C = \frac{WV^2}{127R} \tag{3.2}$$

Where

C = centrifugal force acting on the vehicle (MT)

W = mass of the vehicle (MT)

V = maximum speed of the vehicle (kmph)

R = radius of curve (m)

For railways, Cl. 2.5.3 of Bridge Rule specifies that for broad gauge track, centrifugal force is considered to act at a height of 1.83 m above rail level. Its value is determined as[11]:

$$C = \frac{WV^2}{127R} \tag{3.3}$$

Where

C = centrifugal force acting on the vehicle (MT/m)

W = Equivalent Distributed live load (MT/m)

V = maximum speed of train (kmph)

R = radius of curve (m)

Hence, present provisions in Indian Code for both road and railway bridges provide for consideration of centrifugal force as per calculation of static force.

Euro Code, EN 1991-2:2003 (Cl. 4.4.2) specifies that centrifugal force for road bridges should be calculated as[5]:

$$Q_{tk} = \frac{40Q_{vk}}{r} \tag{3.4}$$

Where

Q_{tk} = centrifugal force acting on the vehicle (kN)

Q_{vk} = Total vertical load of vehicle (kN)

r = radius of curve (m)

Euro Code, EN 1991-2:2003 (Cl. 6.5.1) specifies that centrifugal force for rail bridges should be calculated as[5]:

$$Q_{tk} = \frac{f.Q_{vk}V^2}{127r} \tag{3.5}$$

Where

Q_{tk} = centrifugal force acting on the vehicle (kN)

Q_{vk} = Total vertical load of vehicle (kN)

f = a reduction factor, 1.0 for V ≤ 120 km/h.

r = radius of curve (m)

It is observed that though for road bridges, provisions of Euro code are a bit different than that given in IRC:6, but matches with IRC:6 at a speed of 71 km/h. However for railway bridges, same provisions are there as in Bridge Rule.

3.3 EFFECT OF LATERALLY MOVING LOAD ON PIER

A lateral point load moving on deck along the axis of the bridge is transferred to the piers through bridge deck. Fig. 3.1 shows part of a bridge with 3 piers and 2 superstructures, 20 m each. Fig. 3.2 presents the load on the pier P2 for a lateral load moving from pier P1 to P3.

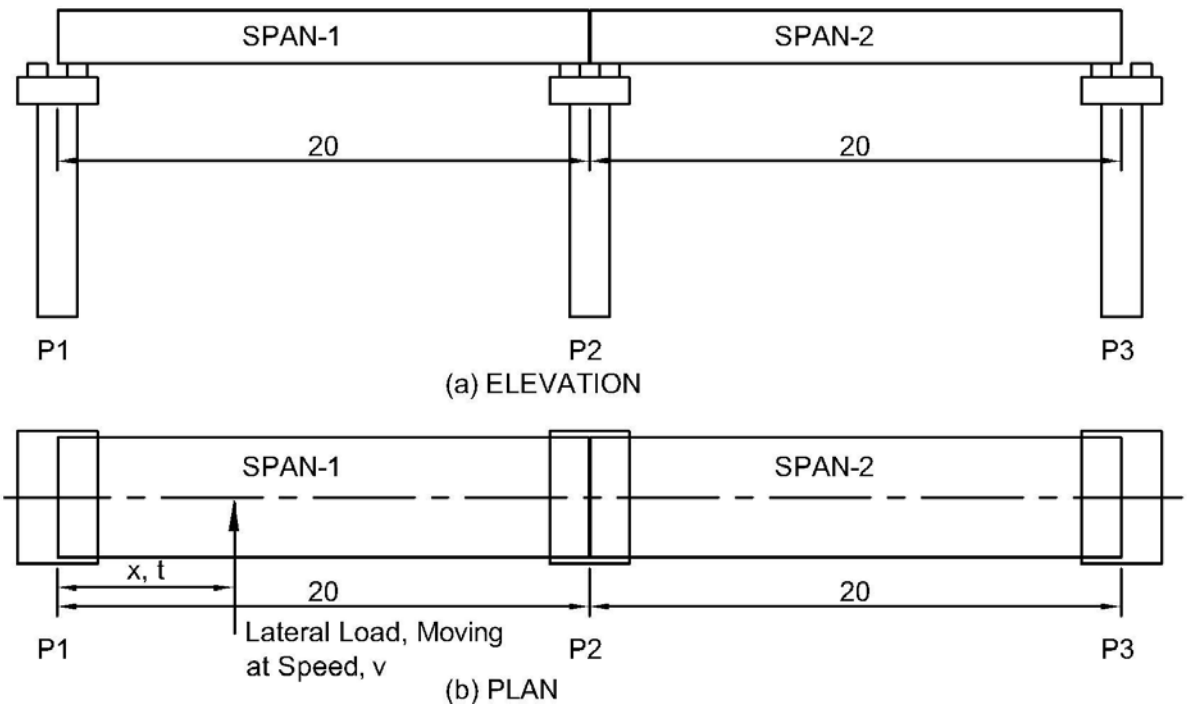


Fig. 3.1 Part of a bridge (3 Piers with 2 Superstructures)

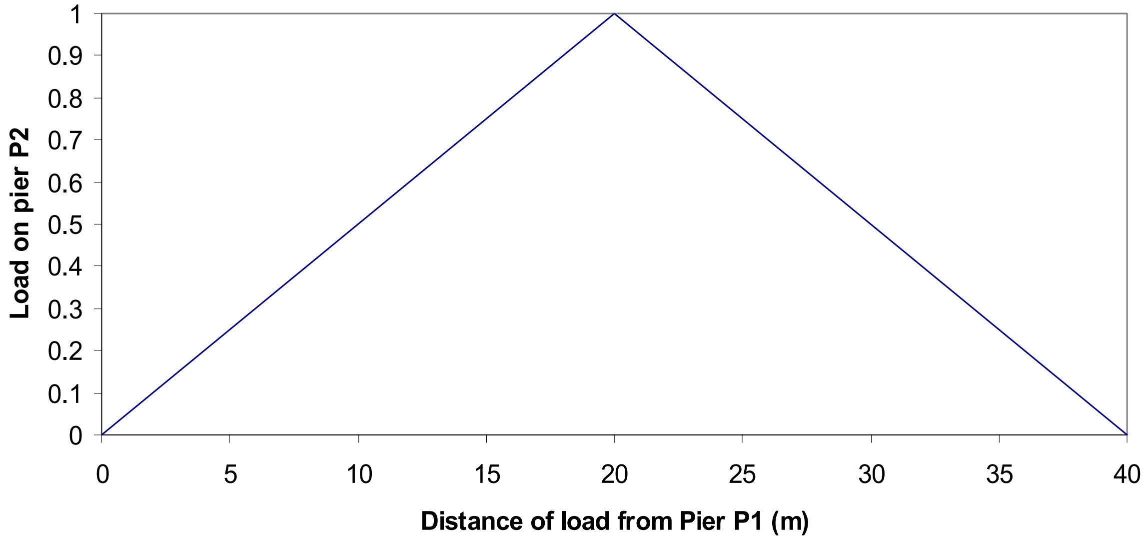


Fig. 3.2 Load on Pier P2 due to a Moving Lateral Load

If load is on pier P1 at time 0 and moving with a speed v m/s then x-axis in the Fig. 3.2 showing the location of point load (x) can be replaced with time (t=x/v) at which point load is at location (x). This suggests that effect of a moving lateral load on pier can be considered as a triangular impulse load, duration of which depends upon the spacing of adjoining piers and speed of the load. This impulse load can be used as load time history for analysis of pier. The analogy can be extended for a series of moving point loads. Fig. 3.3 presents the load on pier P2 due to a train of 10 point loads spaced 8 m apart.

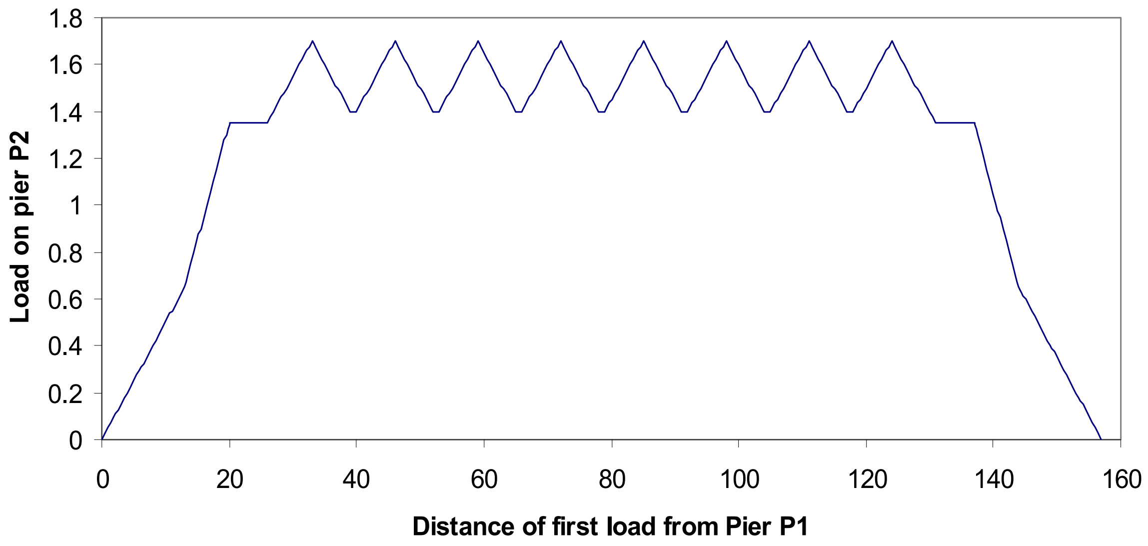


Fig. 3.3 Load on Pier P2 due to Train of Moving Loads

It can be seen that besides constant load, load pattern in Fig. 3.3 has a significant variable load component also which will get converted into periodic load once x-axis is replaced with time as done earlier.

The centrifugal force is transferred to the bridge as moving rail-wheel contact points resulting in a series of moving lateral point loads. Therefore, the loading on bridge due to centrifugal force, when represented as a function of time, should also have a significant harmonic component as well. Bridge superstructure is normally sufficiently rigid in plane of the centrifugal force due to presence of deck slab, thus having a very short natural time period. However, the pier is flexible & also has large concentrated mass of superstructure, pier cap etc. on top of it, hence, has longer fundamental time period. If the frequency of harmonic component of centrifugal force is close to the natural frequency of the pier, there may be a tendency of resonance resulting in large displacement and forces in pier due to centrifugal force and under such conditions, force calculations from static theory may not be reliable.

3.4 IDENTIFICATION OF PARAMETERS FOR STUDY

It has been noted that dynamic response of bridge pier depends upon various factors, e.g., Span Length, Weight of span, Height of pier, Size (dia) of Pier, Speed of train, Train Configuration, Damping etc.

In the present study, Bombardier's MOVIA 8 coach metro train has been chosen, hence, the configuration of train is already fixed. Further, for RCC structures, damping has been specified as 5% by various codes, hence, that too is fixed. Accordingly, in the present study, it is proposed to study the effect of rest of the parameters, viz.,

i. Span Length
ii. Weight of span
iii. Height of pier
iv. Size (Dia) of Pier
v. Speed of train

The range of variation of these parameters will be discussed in subsequent text.

3.5 PROPOSED WORK

In the present work, dynamic analysis of bridges for various bridge configurations having 8-12 m high piers supporting spans ranging from 4-60 m on either side will be carried out for centrifugal force transferred due to passing of a 8 coach metro train at speed varying from 5 m/s to 40 m/s. Weight of span per meter will also be varied for few spans to study the effect of the same. The speed limits of 5 to 40 m/s has been chosen to cover the wider spectrum of trains and operational conditions in future though as per manufacturer's (Bombardier) specification, presently coaches being used in Metro are fit to run upto 85 kmph, i.e., 23.6 m/s.

Pier forces obtained from each set of pier height-span configuration at various speeds using dynamic analysis will be compared with corresponding static forces obtained using present codal provisions. Based on the outcome of dynamic analysis, span lengths and pier heights not resulting in significant dynamic amplification will be categorised as safe for curved application. Recommendations on use of other spans will also be formulated.

CHAPTER-4

LOADING SPECTRA FOR DIFFERENT SPANS DUE TO MOVING TRAIN LOAD

4.1 PREAMBLE

For dynamic analysis of bridge piers, the loading spectra on pier due to moving train is of utmost importance as further analysis depends on the loading spectra. In this chapter, loading spectra for Metro Train will be generated for different span lengths for further use in dynamic analysis of piers. The pattern/ basic characteristics of loading spectra generated will also be discussed.

4.2 CHARACTERISTICS OF PASSENGER TRAIN

Presently different types of coaches are being used in metro trains. Most common coach used by Delhi Metro is Bombardier's MOVIA metro, Fig. 4.1. Salient features of the coach are given in Table 4.1[13].

Fig. 4.1 Bombardier's MOVIA Metro Coach

Table 4.1 Salient features of Bombardier's MOVIA metro Coach

S.N.	Feature	Value	Unit
1	Max. operational speed	85	Km/h
2	Tare weight	42	MT/car
3	Max axle load	17	MT
4	Passengers per car @ 8/sqm	324	Nos
5	Max loaded weight	65	MT
6	Wheel base	2500	mm
7	Bogie distance	15000	mm
8	Car length	22240	mm
9	Car width	3200	mm
10	Floor height (ARL)	1130	mm

(Source: Product Brochure from web site of the manufacturer[13])

The coaches can be arranged in 4, 6 & 8 car train formations. In the analysis, longest 8 coach train configuration has been used so as to have the most critical effect on the bridge.

4.3 SELECTION OF SPAN LENGTHS FOR DYNAMIC ANALYSIS

The bridges in city areas are built with various span lengths considering local as well as functional requirements as per site, usually ranging from 8 to 50 m. In present study, it is proposed to cover the curved bridges with spans ranging from 4-60 m on either side. During the study, it has been noted that loading spectra changes rapidly for shorter spans in comparison to that for longer spans. Accordingly, the spans have been chosen at an interval of 2 m for span range 4-40m and at an interval of 5 m for span range 45-60 m spans.

4.4 LATERAL LOAD TIME HISTORY DUE TO A METRO TRAIN MOVING ON CURVED BRIDGE SPANS

Loading spectra on pier for all 23 curved bridge spans as detailed in 4.3 above has been generated for 8 coach metro train. Moving load simulation has been carried out in STAAD at suitable distance increment size (distance from one position of train on span to next position) for proper discretisation of loading history for various spans as given in Table 4.2.

Table 4.2 Increment Size used for Generating Loading History

S.N.	Span (m)	Increment Size (m)
1	4-12 m @ 2 m interval	0.20
2	14-18 m @ 2 m interval	0.25
3	20-40 m @ 2 m interval	0.50
4	45-60 m @ 5 m interval	0.50

For generating the loading spectra for a span, unit centrifugal force per coach (equally divided among four axles of each coach as 0.25 units / axle) has been applied as moving load. Fig. 4.2 presents the moving load configuration used for generating the loading spectra. Horizontal reaction on pier has been obtained from STAAD analysis for each load position.

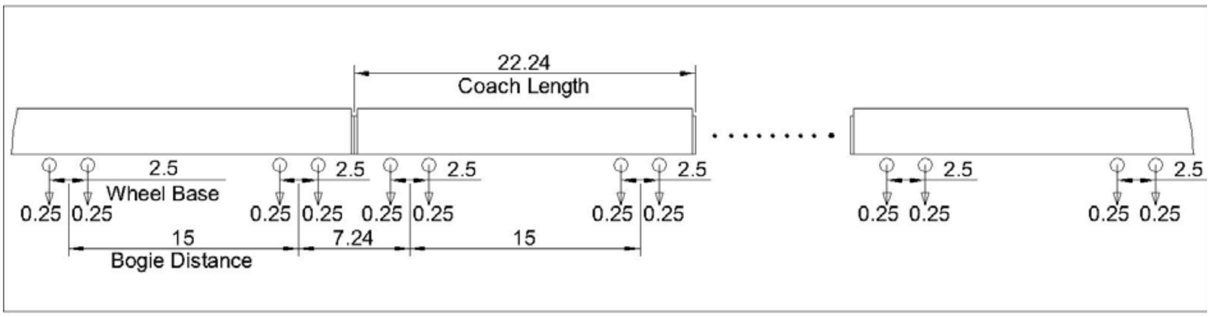


Fig. 4.2 Moving Centrifugal Load Configuration for 8 Coach Metro Train

Fig. 4.3 gives the resultant centrifugal force on pier due to unit force per coach for 4 m span for different placements of train in increment of 0.2 m, as mentioned above. Maximum centrifugal force and Amplitude of dynamic component, i.e., difference of maximum and minimum values of centrifugal force in load history (excluding transition portion in the beginning and at the end) has also been given below the figure for ready reference.

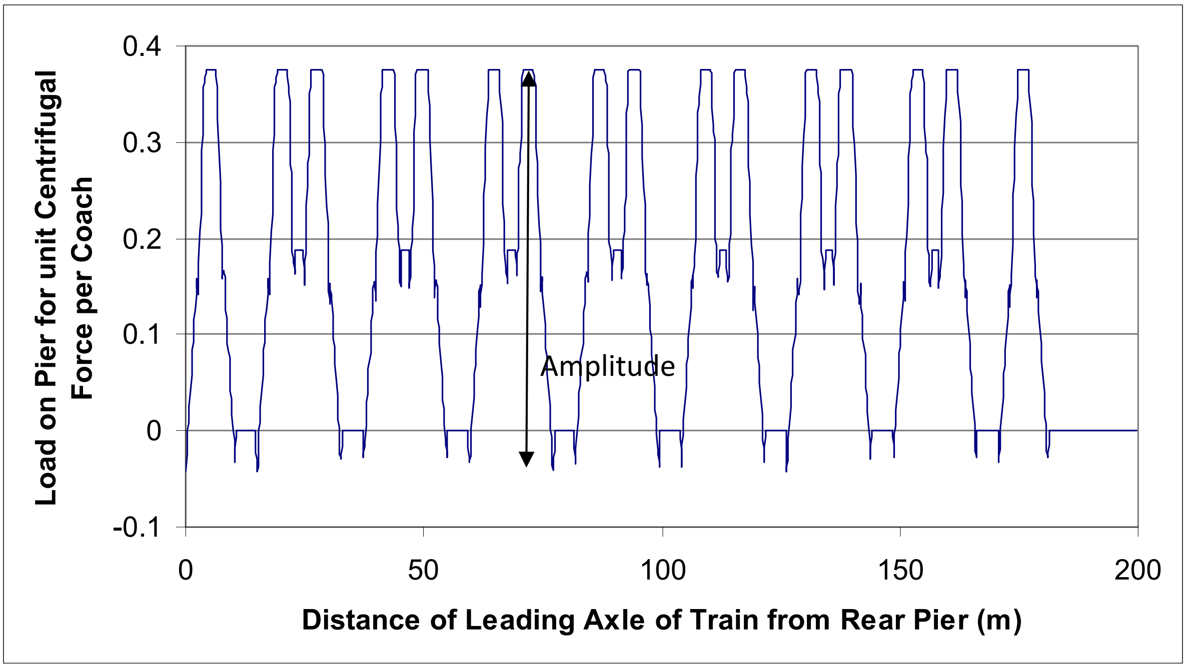


Fig. 4.3 Centrifugal Force on Pier for Different Train Placements on 4 m span

Max CF = 0.375 Amplitude of Dynamic Component = 0.417

Fig 4.4 to Fig.4.25 gives the resultant centrifugal force on pier due to unit force per coach for other spans considered in the study (6 to 40 m @ 2 m and 45 to 60 m @ 5 m interval) for different placements of train in increments as mentioned in Table 4.2. Maximum centrifugal force and Amplitude of dynamic component, as explained earlier, has also been given below each figure for ready reference.

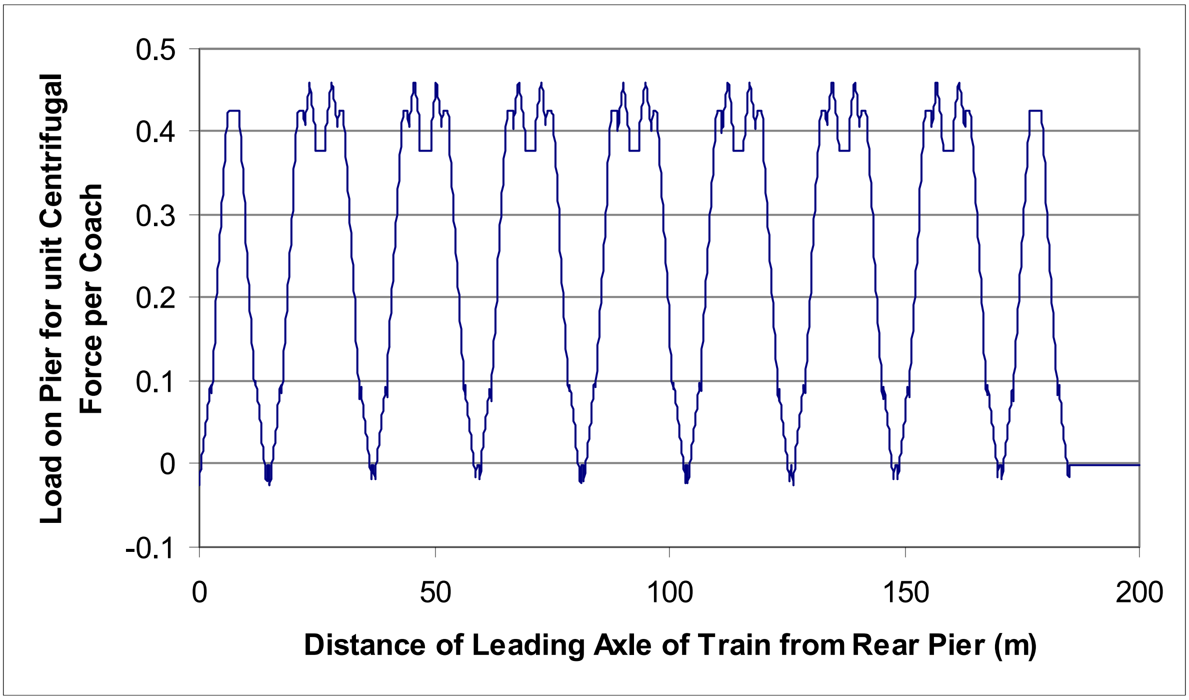


Fig. 4.4 Centrifugal Force on Pier for Different Train Placements on 6 m span

Max CF = 0.458 Amplitude of Dynamic Component = 0.483

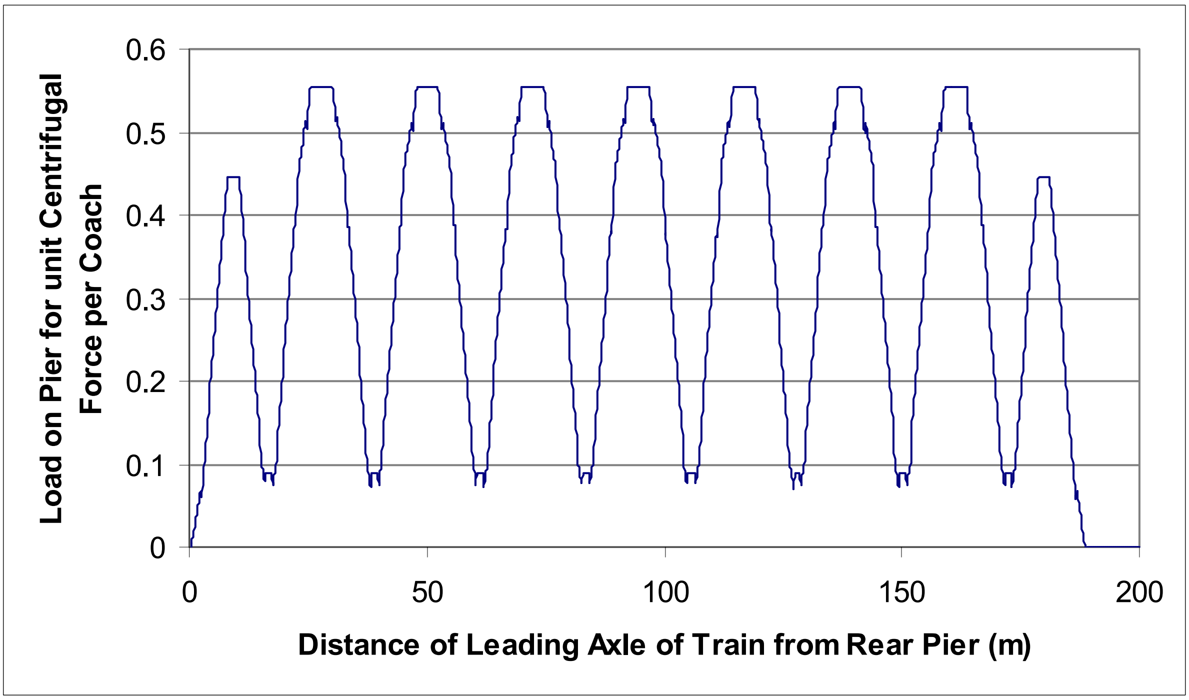


Fig. 4.5 Centrifugal Force on Pier for Different Train Placements on 8 m span

Max CF = 0.554 Amplitude of Dynamic Component = 0.483

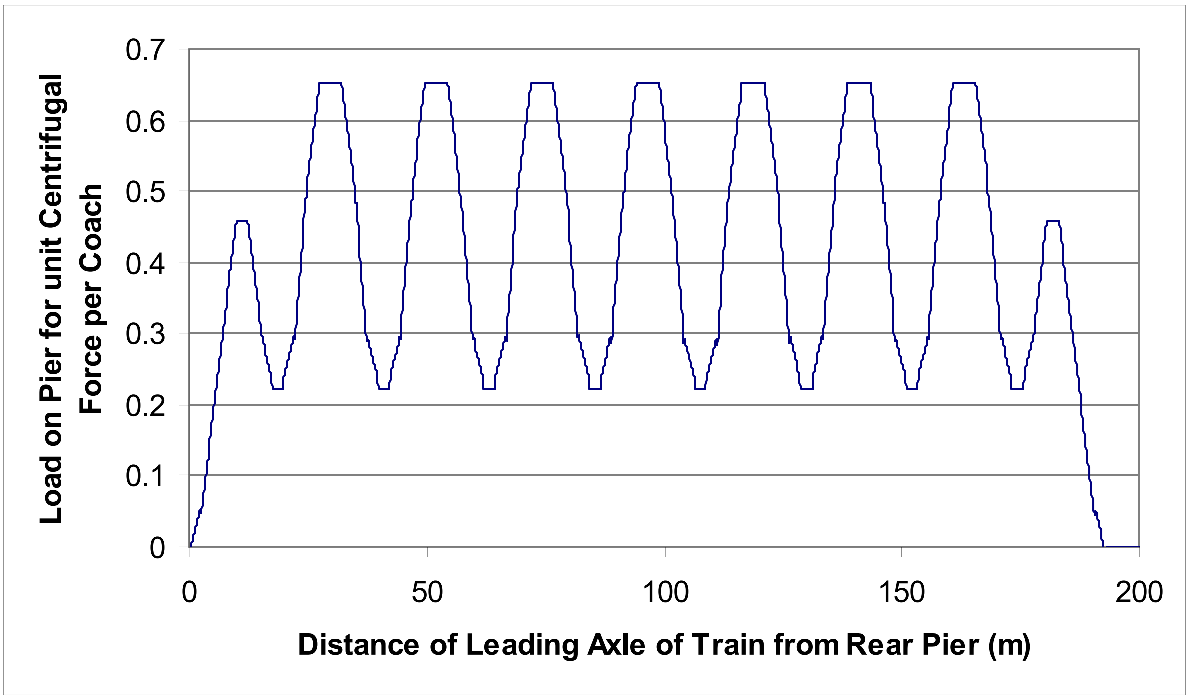


Fig. 4.6 Centrifugal Force on Pier for Different Train Placements on 10 m span

Max CF = 0.653 Amplitude of Dynamic Component = 0.431

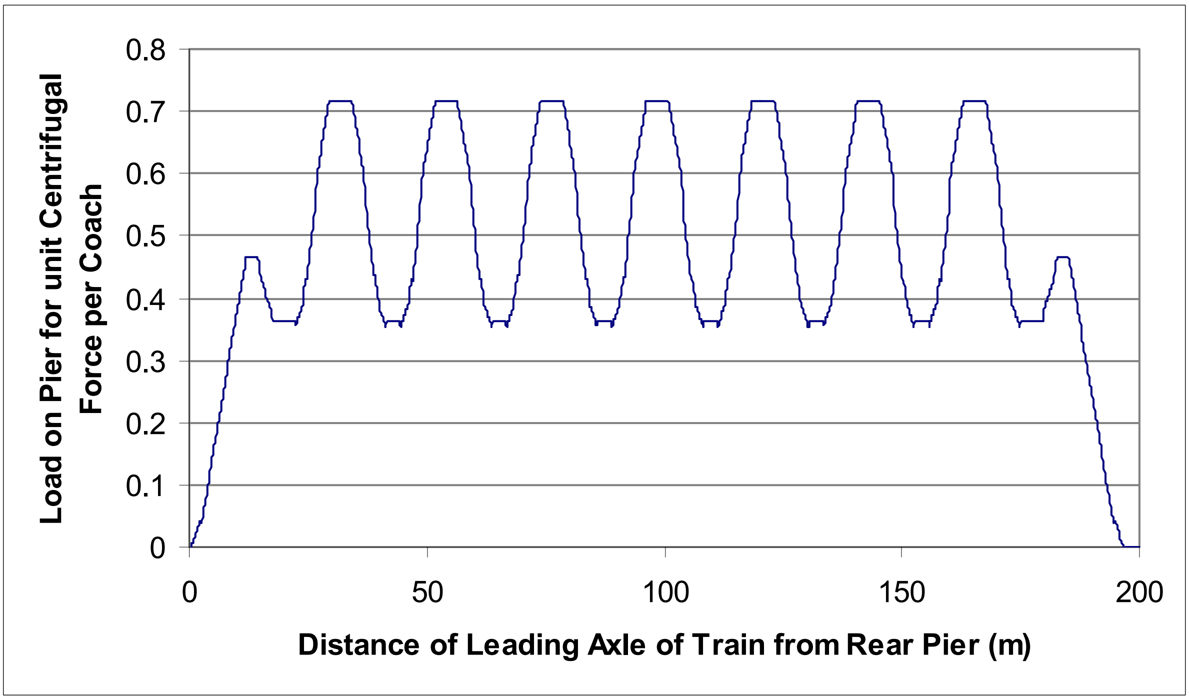


Fig. 4.7 Centrifugal Force on Pier for Different Train Placements on 12 m span

Max CF = 0.716 Amplitude of Dynamic Component = 0.368

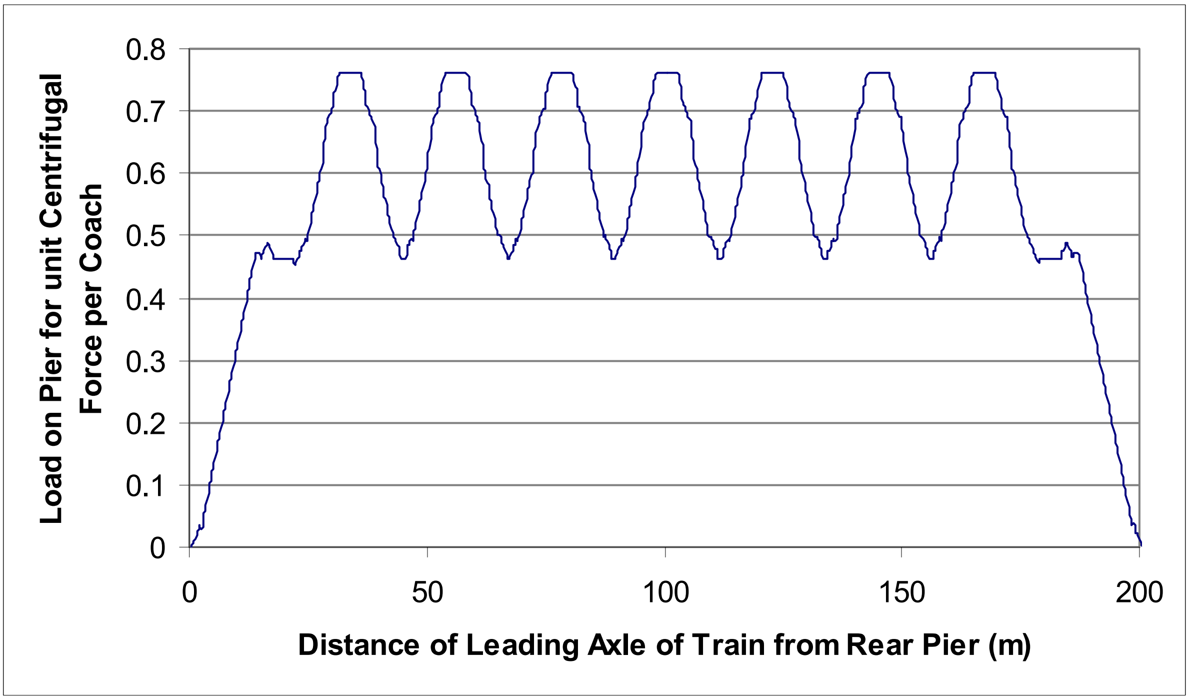


Fig. 4.8 Centrifugal Force on Pier for Different Train Placements on 14 m span

Max CF = 0.760 Amplitude of Dynamic Component = 0.298

Fig. 4.9 Centrifugal Force on Pier for Different Train Placements on 16 m span

Max CF = 0.799 Amplitude of Dynamic Component = 0.215

Fig. 4.10 Centrifugal Force on Pier for Different Train Placements on 18 m span

Max CF = 0.853 Amplitude of Dynamic Component = 0.102

Fig. 4.11 Centrifugal Force on Pier for Different Train Placements on 20 m span

Max CF = 0.921 Amplitude of Dynamic Component = 0.039

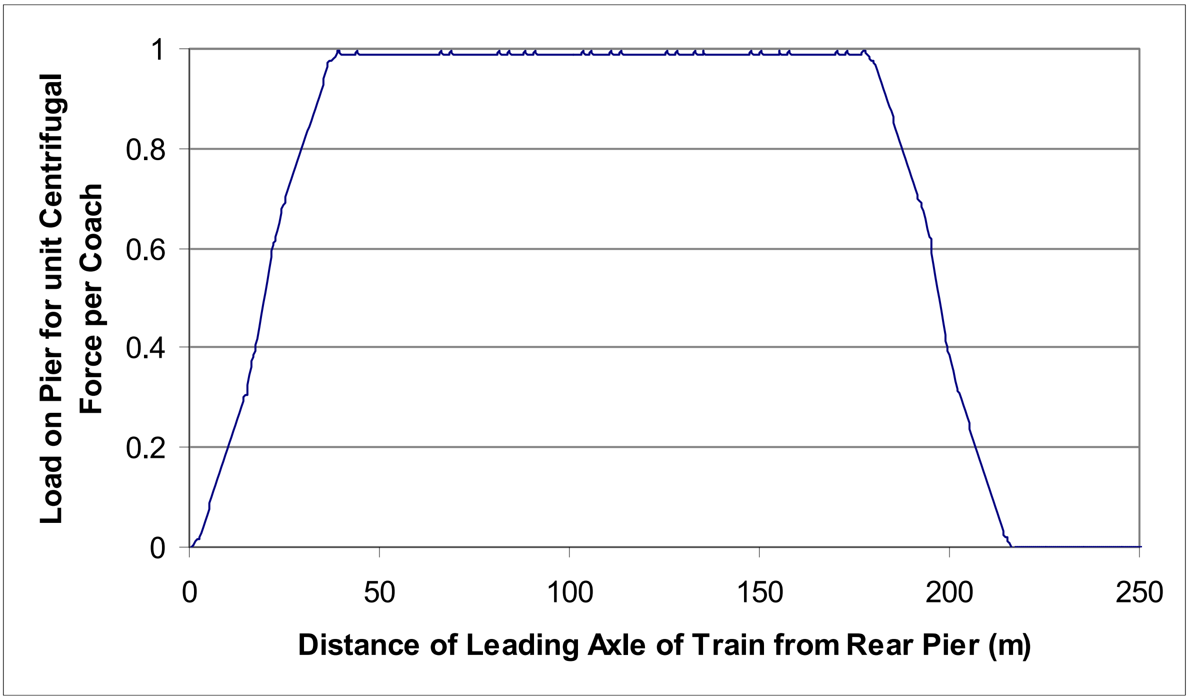


Fig. 4.12 Centrifugal Force on Pier for Different Train Placements on 22 m span

Max CF = 0.996 Amplitude of Dynamic Component = 0.011

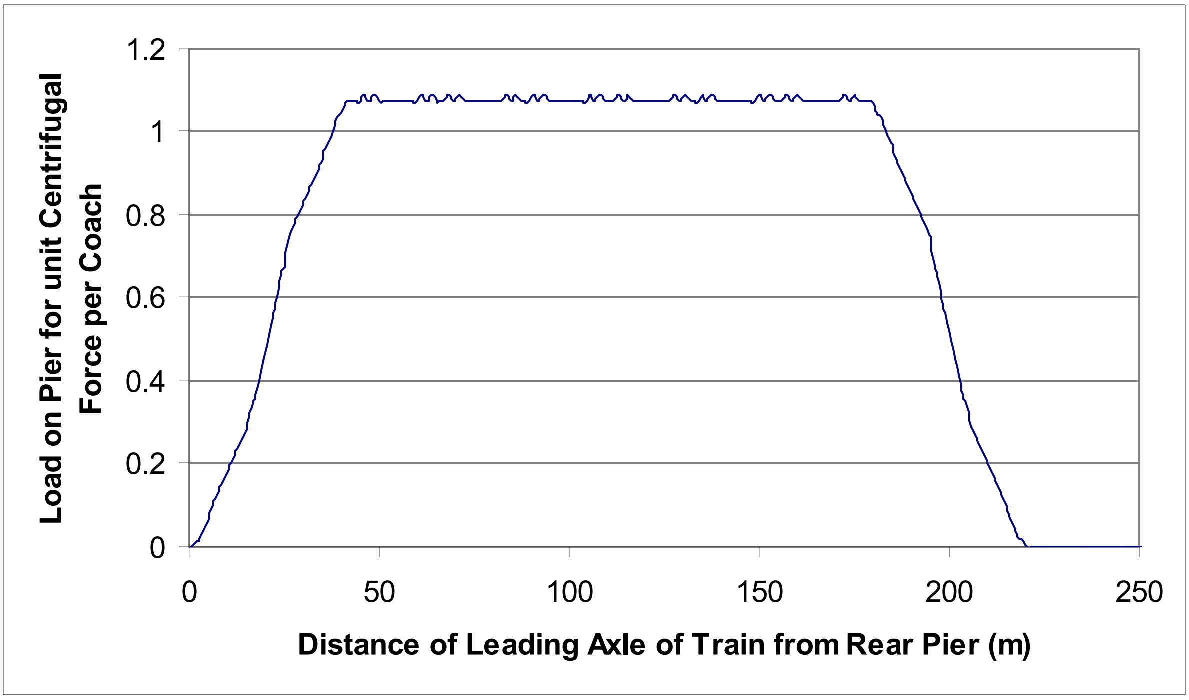


Fig. 4.13 Centrifugal Force on Pier for Different Train Placements on 24 m span

Max CF = 1.089 Amplitude of Dynamic Component = 0.018

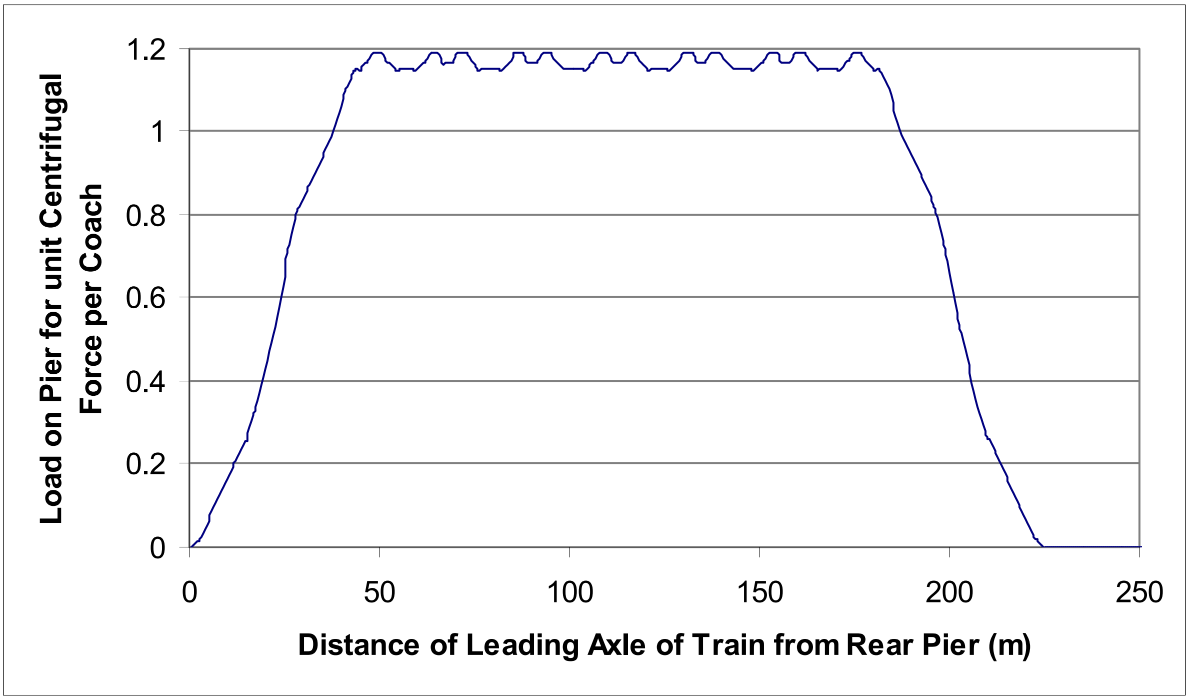


Fig. 4.14 Centrifugal Force on Pier for Different Train Placements on 26 m span

Max CF = 1.191 Amplitude of Dynamic Component = 0.045

Fig. 4.15 Centrifugal Force on Pier for Different Train Placements on 28 m span

Max CF = 1.291 Amplitude of Dynamic Component = 0.082

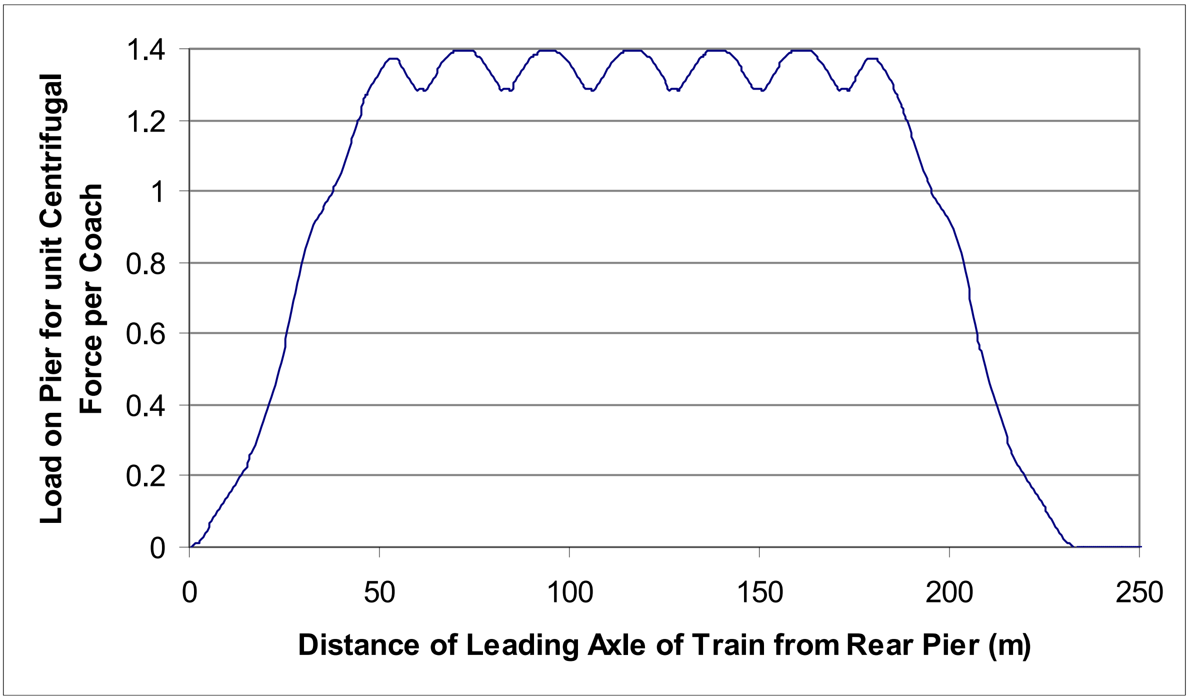


Fig. 4.16 Centrifugal Force on Pier for Different Train Placements on 30 m span

Max CF = 1.393 Amplitude of Dynamic Component = 0.112

Fig. 4.17 Centrifugal Force on Pier for Different Train Placements on 32 m span

Max CF = 1.497 Amplitude of Dynamic Component = 0.125

Fig. 4.18 Centrifugal Force on Pier for Different Train Placements on 34 m span

Max CF = 1.588 Amplitude of Dynamic Component = 0.121

Fig. 4.19 Centrifugal Force on Pier for Different Train Placements on 36 m span

Max CF = 1.669 Amplitude of Dynamic Component = 0.115

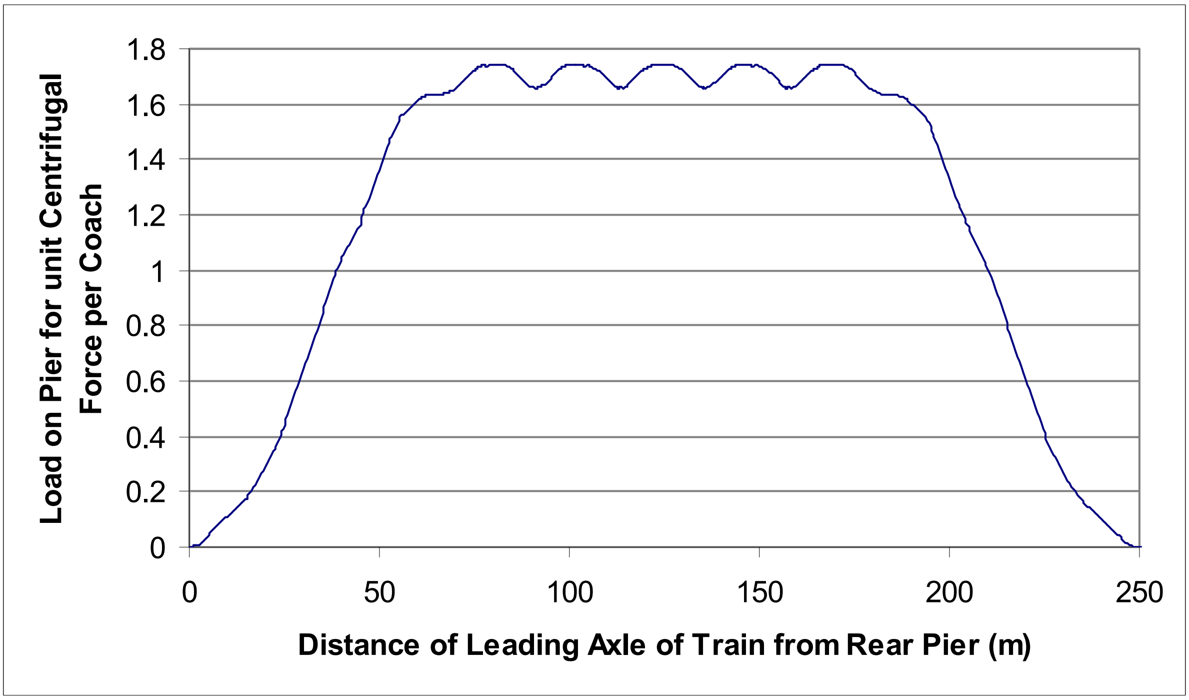


Fig. 4.20 Centrifugal Force on Pier for Different Train Placements on 38 m span

Max CF = 1.742 Amplitude of Dynamic Component = 0.086

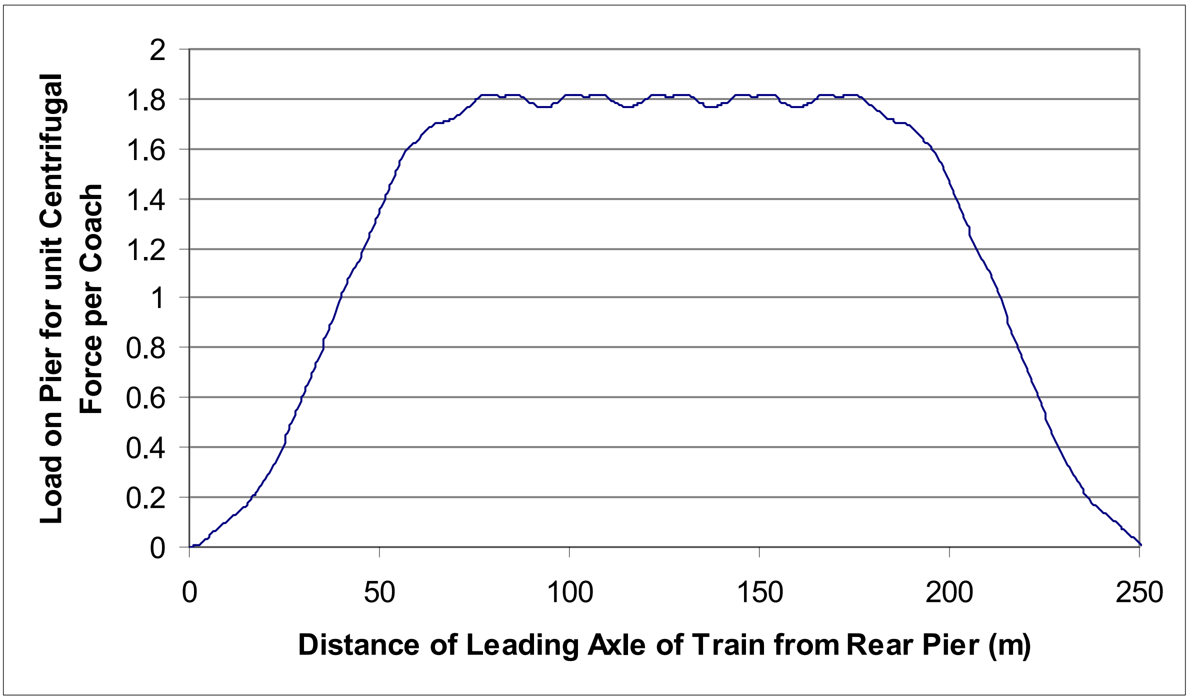


Fig. 4.21 Centrifugal Force on Pier for Different Train Placements on 40 m span

Max CF = 1.818 Amplitude of Dynamic Component = 0.048

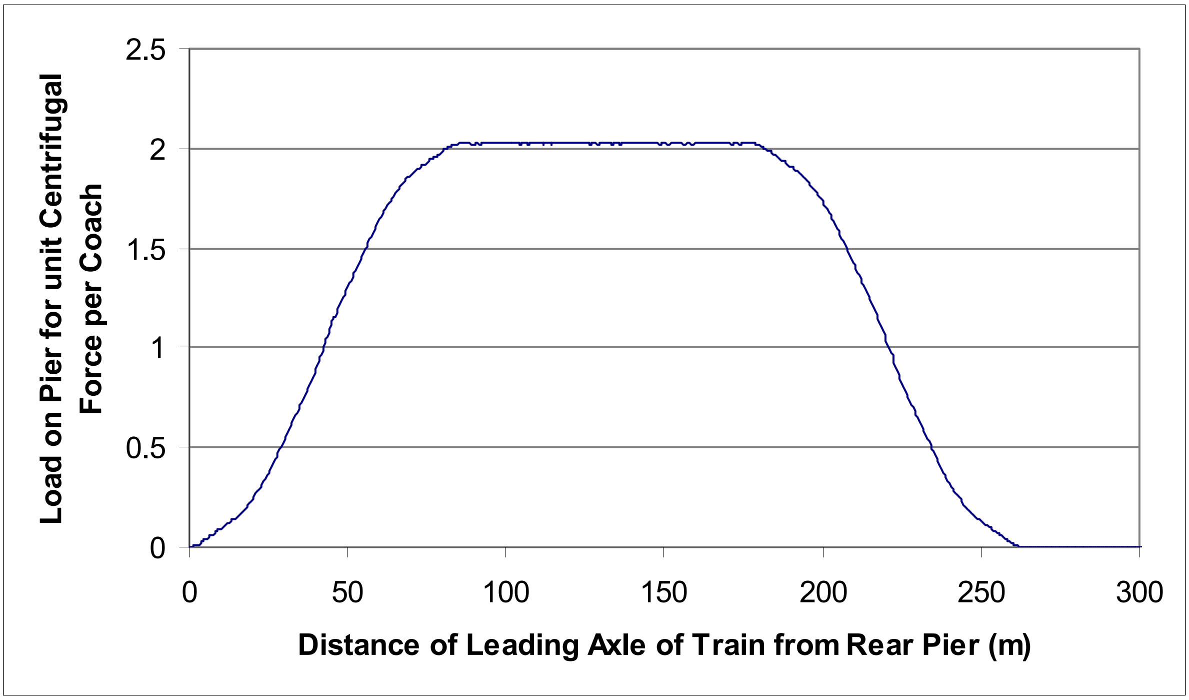


Fig. 4.22 Centrifugal Force on Pier for Different Train Placements on 45 m span

Max CF = 2.024 Amplitude of Dynamic Component = 0.003

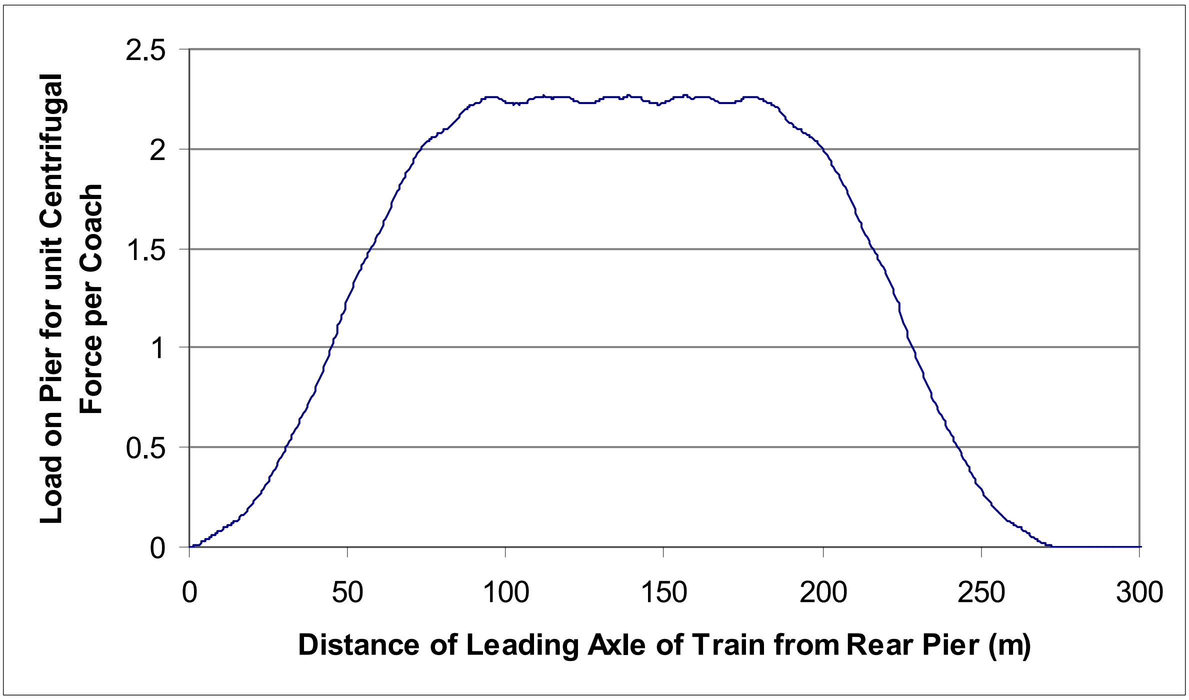


Fig. 4.23 Centrifugal Force on Pier for Different Train Placements on 50 m span

Max CF = 2.265 Amplitude of Dynamic Component = 0.042

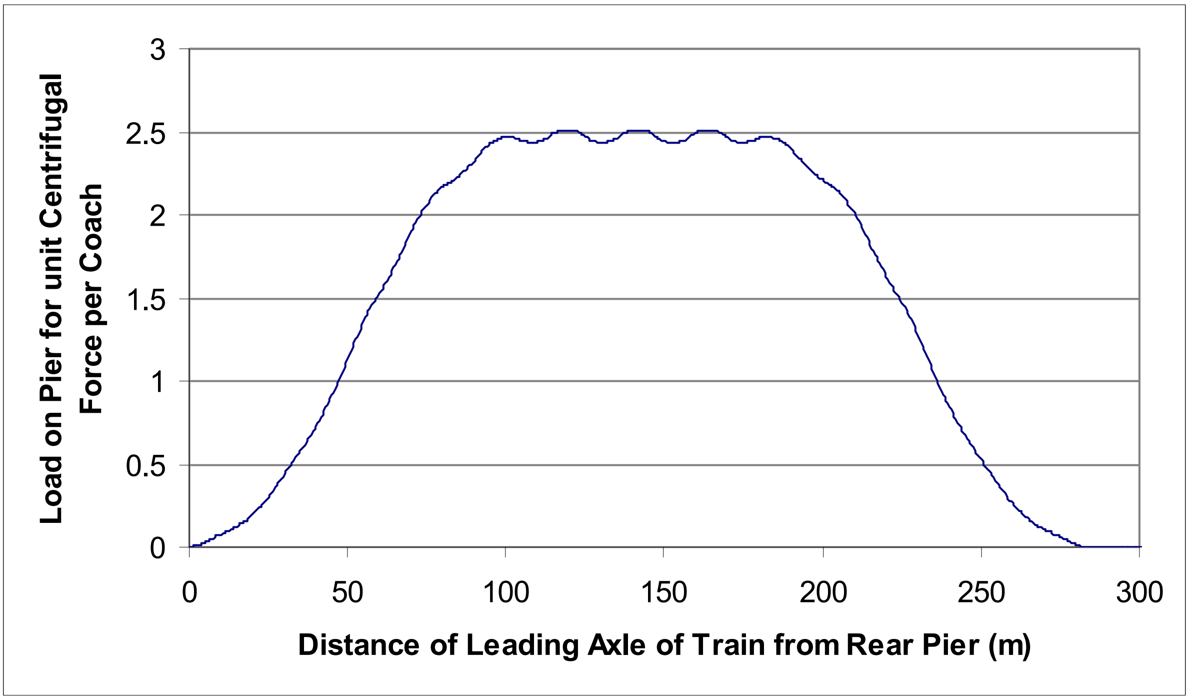


Fig. 4.24 Centrifugal Force on Pier for Different Train Placements on 55 m span

Max CF = 2.508 Amplitude of Dynamic Component = 0.072

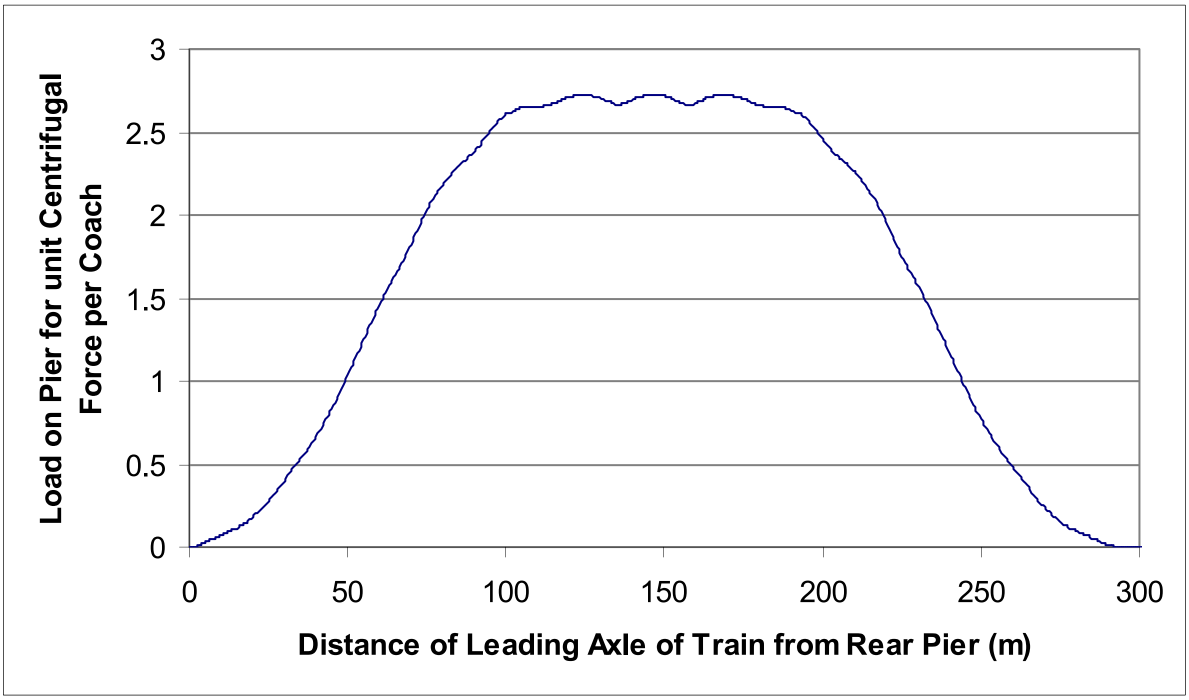


Fig. 4.25 Centrifugal Force on Pier for Different Train Placements on 60 m span

Max CF = 2.719 Amplitude of Dynamic Component = 0.055

4.5 DISCUSSION ON LOADING HISTORY FOR VARIOUS SPANS

Maximum centrifugal force and amplitude of dynamic component from loading history of all the spans (given in Fig. 4.3 to Fig. 4.25) as mentioned below each figure have been summarized in Table 4.3. Average centrifugal force for each span has also been calculated and mentioned in Table 4.3 for comparison.

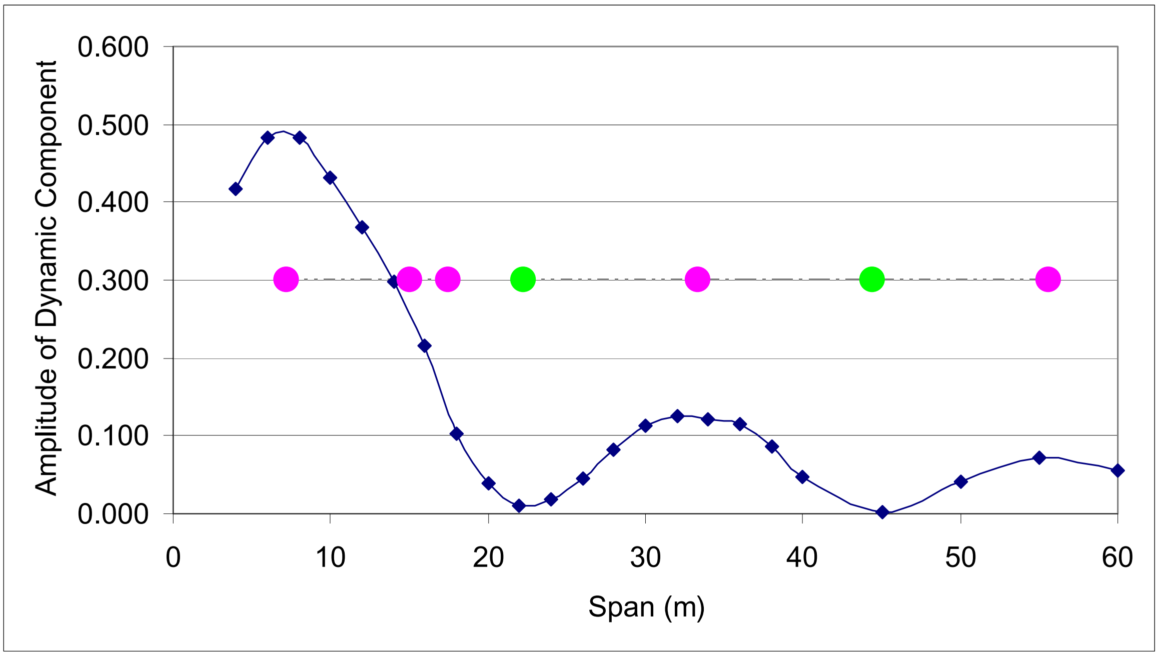
Table 4.3 Pier Reaction due to Unit Centrifugal Force per coach for different Spans

Span	Maximum	Average	Amplitude of Dynamic Component
4	0.375	0.175	0.417
6	0.458	0.269	0.483
8	0.554	0.360	0.483
10	0.653	0.449	0.431
12	0.716	0.537	0.368
14	0.760	0.624	0.298
16	0.799	0.720	0.215
18	0.853	0.809	0.102
20	0.921	0.899	0.039
22	0.996	0.990	0.011
24	1.089	1.079	0.018
26	1.191	1.169	0.045
28	1.291	1.259	0.082
30	1.393	1.349	0.112
32	1.497	1.438	0.125
34	1.588	1.529	0.121
36	1.669	1.619	0.115
38	1.742	1.709	0.086
40	1.818	1.799	0.048
45	2.024	2.024	0.003
50	2.265	2.247	0.042
55	2.508	2.469	0.072
60	2.719	2.698	0.055

Fig. 4.26 presents the variation of amplitude of dynamic component of pier reaction due to unit centrifugal force per coach for different spans. To analyze the pattern obtained, some critical distances (as given in Table 4.4) have also been marked in the figure.

Table 4.4 Critical distances in Train Formation

S.N.	Description	Distance (m)	Marked As
	Basic Bogie/Wheel Distances		
1.	Inter coach Bogie Distance	7.24	●
2.	Intra coach Bogie Distance	15	●
3.	Outer Wheel distance in one coach	17.5	●
	Multiples of half coach length		
4.	1.5 Coach Length	33.36	●
5.	2.5 Coach length	55.6	●
	Multiples of full coach length		
6.	1 Coach length	22.24	●
7.	2 Coach length	44.48	●



● ● Critical distances in train formation (Table 4.4)

Fig. 4.26 Amplitude of Dynamic Component of Pier Reaction due to Unit Centrifugal Force per coach for different Spans

It has been observed that spans in multiple of full coach length have negligible dynamic component whereas other spans have significant dynamic component. Peaks are observed for span length equal to inter-coach bogie distance and for odd multiple of half coach length. The effect of the same on pier response will be further studied after dynamic analysis of piers.

Due to unit centrifugal force per coach considered in generating the load history, it is speed independent. To obtain the load-time history for dynamic analysis of pier due to centrifugal force, the load ordinate will subsequently be scaled for actual value of centrifugal force per coach at desired speed. The load position will also be converted into time by dividing load position coordinate by train speed.

4.6 CONCLUSION

Loading history for 8 coach metro trains moving on various spans have been generated for further dynamic analysis of piers. The loading history generated is independent of train speed. Speed specific load-time history can be generated by scaling load (Y) ordinate of spectra for actual value of centrifugal force per coach at speed under consideration and converting load position (X ordinate) into time by dividing load position coordinate by train speed.

It has been noted that loading spectra varies from span to span. It has been observed that for span lengths close to multiples of coach length (22m and 44 m), loading spectra has negligible dynamic component. For other spans, it is observed to have significant dynamic component which may alter the pier response in dynamic analysis.

CHAPTER-5

ANALYSIS OF CURVED BRIDGES OF DIFFERENT SPANS AND PIER HEIGHTS

5.1 PREAMBLE

For dynamic analysis of bridge piers, the essentials inputs include loading history and bridge parameters. The train definition and loading history on pier due to moving train has already been developed in chapter-4. In this chapter, the bridge parameters identified for study in para 3.4 of chapter 3 will be defined for further use in analysis. The dynamic analysis of bridge piers will be carried out using these parameters and loading history and results obtained will be analysed. On the basis of outcome of analysis of results, recommendations on use of spans will also be finalised.

5.2 CURVED BRIDGE PARAMETERS

5.2.1 Bridge Configuration

In present study, curved bridges with 8, 10 and 12 m high piers supporting spans ranging from 4-60 m on either side (4-40 m spans @ 2 m increment and 45-60 m spans @ 5 m increment) have been considered. For the purpose of parametric study, various pier height and span configurations have been combined into 5 groups. Typical pier height & dia and cross section of span for each group has been chosen to commensurate with the span range considered in the group. Salient features of the span and pier data for various bridge groups are given in Table 5.1.

The parameters of track and dimensions of pier cap for the bridge used in analysis are given in Table 5.2.

Table 5.1 Salient Features of the Bridge Groups Considered for Analysis

S.N.	Feature	Unit	Group-1	Group-2	Group-3	Group-4	Group-5
A	**Super structure**						
1	Span c/c of pier	m	4-30 @ 2	20-38 @ 2	30, 32, 36, 40, 45, 50, 55, 60	30, 32, 36, 40, 45, 50, 55, 60	30, 32, 36, 40, 45, 50, 55, 60
2	Cross sectional area of span	sqm	2.10	2.10	3.30	3.30	3.30
3	Weight of Span	Ton/m	5.25	5.25	8.25	8.25	8.25
4	Height of CG of super structure above its base	m	0.50	0.50	0.50	0.50	0.50
5	Height of pedestal + bearing	m	0.40	0.40	0.40	0.40	0.40
6	No of superstructure/pier cap		2	2	2	2	2
B	**Pier**						
1	Dia	m	1.50	1.80	2.20	2.20	2.20
2	Height up to top of pier cap	m	8.00	8.00	8.00	10.00	12.00
3	Concrete, fck	MPa	35	35	35	35	35
4	Ec	MPa	30000	30000	30000	30000	30000

Table 5.2 General Features of the Bridge used for Analysis

S.N.	Feature	Value	Unit
A	**Track**		
1	Height of Track+super structure	1.200	m
2	Weight of track	3.000	ton/m
3	No of tracks	2	
4	radius of curvature	360.000	m
B	**Pier Cap**		
1	Average length	9.000	m
2	Average width	3.000	m
3	Average thickness	1.250	m
4	Volume	33.750	cum
5	Weight	84	ton
C	**Damping**	**5**	**%**

For the analysis, one pier with two equal adjoining spans has been considered. Fig. 5.1 gives the typical layout of the bridge for 28 m span. As the mass of superstructure and pier cap is practically lumped on top of pier and self weight of pier is much less in comparison to mass on top, the pier along with spans has been modelled as single degree of freedom system. The properties of this system will be discussed in the subsequent text.

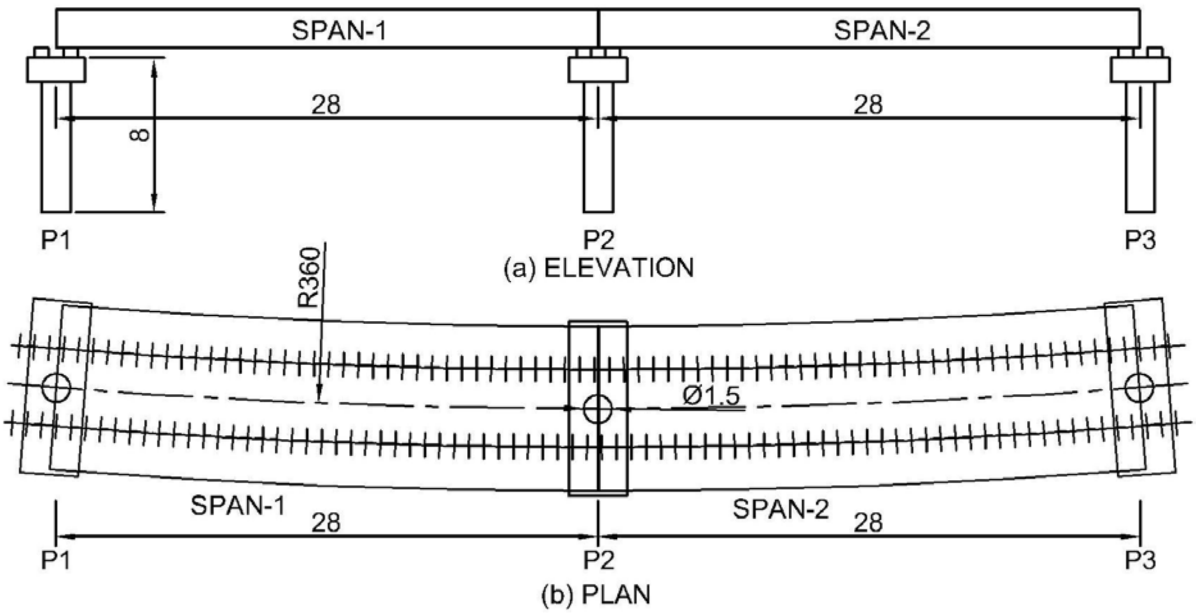


Fig. 5.1 Typical Configuration of the Curved Bridge used in Analysis

5.2.2 Dynamic Properties of the pier

For calculating dynamic properties of the bridge pier + superstructure + Live load system; mass, stiffness and damping have been considered as follows:

5.2.2.1 Mass of bridge

The maximum acceleration of a structure is inversely proportional to the mass of the bridge structure at resonance. The maximum dynamic load effects are likely to occur at resonance peaks, where a multiple of the frequency of loading coincide with natural frequency of the structure. Any underestimation of the mass will overestimate the natural frequency of the structure, and therefore overestimate the traffic speed at which resonance occurs. Therefore, realistic calculation of lumped mass of pier and its CG is essential. Hence, the mass of superstructure along with mass of track, live load, pier cap and half mass of pier has been lumped at top of pier. CG of combined mass has been calculated duly considering their height (Fig. 5.2) and effective pier height has been considered upto that point.

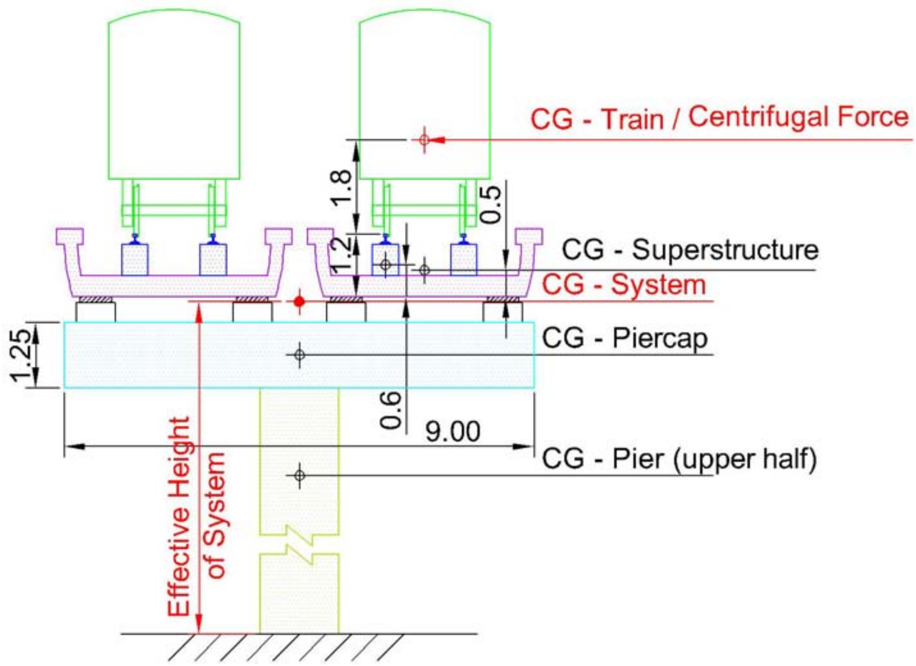


Fig. 5.2 CG of elements of System with Centrifugal Force

5.2.2.2 Stiffness of bridge pier

The stiffness of the bridge structure has an influence on the dynamic effects. Any overestimation of the bridge stiffness will overestimate the natural frequency of the structure and the traffic speed, at which resonance occurs. It has been noted that point of application of centrifugal force (at CG of moving trains) is above then that of CG of system (train + track + superstructure + pier cap + pier), as shown in Fig. 5.2. Hence, the deflection of pier due to centrifugal force when applied at CG of train will be more than that if applied at CG of system leading to reduction in effective stiffness of pier for centrifugal force. Hence, to calculate the stiffness of piers, a unit load has been applied at CG of train (as centrifugal force acts at that point) and corresponding deflection at effective CG of combined mass of system has been calculated. The stiffness of pier has then been calculated as inverse of deflection so obtained. The approach is similar to dynamic condensation suggested by Paz[8]. As the bridge piers are under heavy compression due to dead load of superstructure, the value of the stiffness of pier has been determined using uncracked moment of inertia of pier.

5.2.2.3 Damping of bridge

Damping is a property of building material and state of structures, for example presence of cracks etc. Cl. 219.5.1 of IRC:6 specifies the damping for bridges made with different materials. For RCC elements, a damping of 5% has been specified[7] and the same has been used in analysis. Using Lumped mass, M, damping coefficient has been calculated using Eqn. 2.2.

Using the methodology presented above, system properties required for dynamic analysis of pier (Lumped mass, M; Stiffness of pier, K; Time period, T) for all spans mentioned in Table 5.1 have been given in Table 5.3 to Table 5.7 of groups 1 to 5 respectively. The same will be further used in dynamic analysis.

Table 5.3 System Properties for spans of Group-1

Dia of Pier	= 1.5 m	EI of pier	= 7455147 kN.m^2
Height of pier	= 8.0 m	Height of CG of train	= 11.4 m
Weight of span/m	= 5.25 MT/m		

S.N.	Span (m)	Mass, M (MT)	Eff. Height of Pier (m)	Stiffness, K (kN/m)	Time Period, T (Sec)
1.	4	180	7.88	27380	0.51
2.	6	219	8.13	25947	0.58
3.	8	257	8.31	25022	0.64
4.	10	296	8.44	24375	0.70
5.	12	335	8.54	23898	0.75
6.	14	374	8.62	23532	0.80
7.	16	413	8.68	23242	0.84
8.	18	452	8.74	23006	0.89
9.	20	490	8.78	22812	0.93
10.	22	529	8.82	22647	0.97
11.	24	568	8.86	22508	1.00
12.	26	607	8.88	22387	1.04
13.	28	646	8.91	22282	1.08
14.	30	685	8.93	22189	1.11

Table 5.4 System Properties for spans of Group-2

Dia of Pier	= 1.8 m	EI of pier	= 15458992 kN.m^2
Height of pier	= 8.0 m	Height of CG of train	= 11.4 m
Weight of span/m	= 5.25 MT/m		

S.N.	Span (m)	Mass, M (MT)	Eff. Height of Pier (m)	Stiffness, K (kN/m)	Time Period, T (Sec)
1.	20	498	8.71	47975	0.64
2.	22	537	8.75	47583	0.67
3.	24	576	8.79	47248	0.70
4.	26	615	8.82	46959	0.72
5.	28	654	8.85	46707	0.75
6.	30	693	8.88	46485	0.77
7.	32	731	8.90	46288	0.79
8.	34	770	8.92	46113	0.82
9.	36	809	8.94	45955	0.84
10.	38	848	8.96	45812	0.86

Table 5.5 System Properties for spans of Group-3

Dia of Pier	= 2.2 m	EI of pier	= 34497044 kN.m^2
Height of pier	= 8.0 m	Height of CG of train	= 11.4 m
Weight of span/m	= 8.25 MT/m		

S.N.	Span (m)	Mass, M (MT)	Eff. Height of Pier (m)	Stiffness, K (kN/m)	Time Period, T (Sec)
1.	30	885	8.81	104989	0.58
2.	32	936	8.83	104563	0.60
3.	36	1038	8.87	103845	0.63
4.	40	1139	8.90	103262	0.66
5.	45	1266	8.93	102670	0.70
6.	50	1394	8.96	102191	0.74
7.	55	1521	8.98	101795	0.77
8.	60	1648	9.00	101462	0.81

Table 5.6 System Properties for spans of Group-4

Dia of Pier = 2.2 m

EI of pier = 34497044 kN.m^2

Height of pier = 10.0 m

Height of CG of train = 13.4 m

Weight of span/m = 8.25 MT/m

S.N.	Span (m)	Mass, M (MT)	Eff. Height of Pier (m)	Stiffness, K (kN/m)	Time Period, T (Sec)
1.	30	895	10.71	61209	0.76
2.	32	945	10.73	60957	0.79
3.	36	1047	10.78	60530	0.83
4.	40	1149	10.82	60183	0.87
5.	45	1276	10.86	59831	0.92
6.	50	1403	10.89	59545	0.97
7.	55	1530	10.92	59309	1.02
8.	60	1657	10.94	59110	1.06

Table 5.7 System Properties for spans of Group-5

Dia of Pier = 2.2 m

EI of pier = 34497044 kN.m^2

Height of pier = 12.0 m

Height of CG of train = 15.4 m

Weight of span/m = 8.25 MT/m

S.N.	Span (m)	Mass, M (MT)	Eff. Height of Pier (m)	Stiffness, K (kN/m)	Time Period, T (Sec)
1.	30	904	12.58	38878	0.96
2.	32	955	12.62	38711	0.99
3.	36	1057	12.68	38429	1.05
4.	40	1158	12.72	38199	1.10
5.	45	1285	12.77	37965	1.16
6.	50	1413	12.81	37775	1.22
7.	55	1540	12.84	37618	1.28
8.	60	1667	12.87	37486	1.33

5.3 LATERAL LOAD TIME HISTORY DUE TO A TRAIN MOVING ON A CURVED BRIDGE

Loading history for 8 coach metro train moving on various spans have been generated (given in chapter 4) as a function of distance of leading axle of train from rear pier for further dynamic analysis of piers. Due to unit centrifugal force per coach considered in generating the load history, it is speed independent. To obtain the load-time history for dynamic analysis of pier due to the centrifugal force corresponding to speed under consideration, the load ordinate is required to be scaled for actual value of centrifugal force per coach at desired speed. The load position is also required to be converted into time by dividing load position coordinate by train speed.

The actual centrifugal force on a coach for any speed of train can be calculated using Eqn. 3.1 as

$$C = Mv^2/r \qquad (5.1)$$

where,

C = Centrifugal force per coach (N)

M = mass of the coach (kg)

= 65000 kg for Bombardier's MOVIA Coach adopted for this study

v = speed of the train (m/s)

= 5-40 m/s, as considered in the study for different cases

r = radius of curvature of track (m)

= 360 m adopted in this study,

or

C = $0.18056\ v^2$ kN

Lateral load on pier due to unit centrifugal force per coach for various spans has been plotted in Fig. 4.3 to Fig. 4.25. Centrifugal force-Time history for these spans can be calculated using Equation 5.1 for various speeds of train. Fig. 5.3 presents a typical load-time history for 18 m span at a speed of 20 m/s. It has been obtained by scaling Y axis of Fig. 4.10 (the load on pier) by a factor of $65000 \times 20^2/360 = 72222$ N or 72.222 kN and by dividing X axis values (the distance of leading axle from rear pier) by speed of train, i.e., 20 m/s.

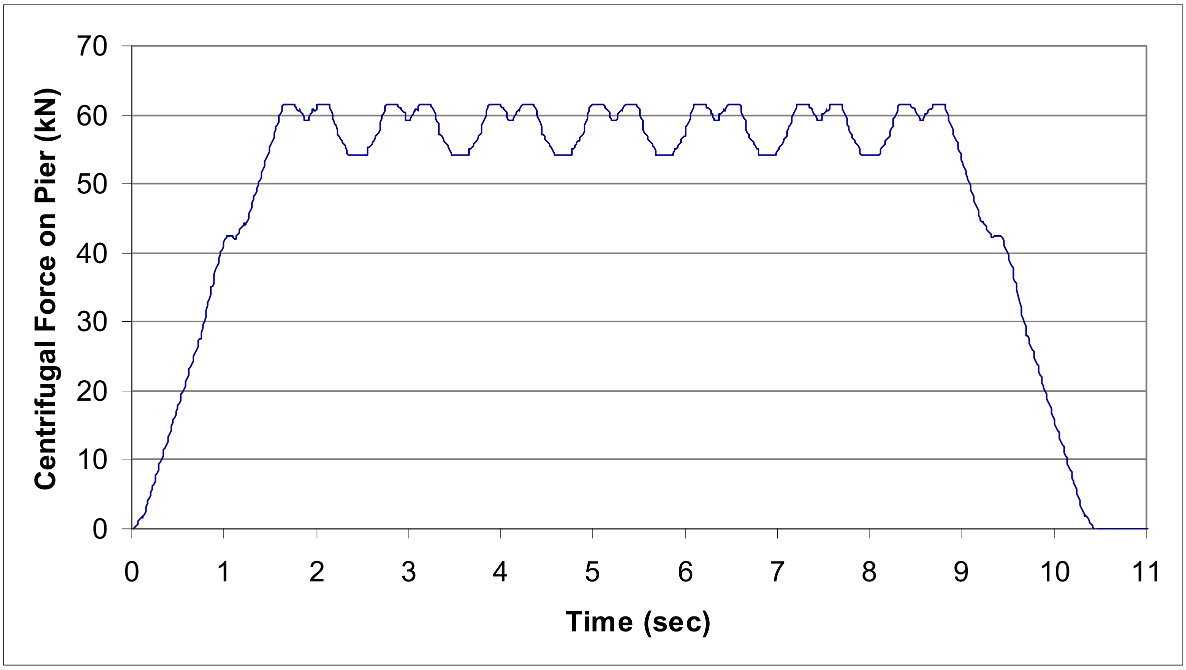


Fig. 5.3 Load-Time History for 18 m Span at Speed of 20 m/s

It has been noted that Fig. 5.3 is similar to Fig. 4.10 except for scales. Load-time histories for other spans and speeds have been obtained similarly. The above mentioned conversion eliminates the need of plotting Load-time history for all spans as the pattern can easily be visualised from Fig. 4.3 to Fig. 4.25.

5.4 DYNAMIC ANALYSIS OF BRIDGE PIER FOR CENTRIFUGAL FORCE

Loading history for unit centrifugal force per coach due to metro train moving on various spans has been developed in Chapter-4. The bridge parameters required for dynamic analysis have been defined in Para 5.2 above. However, to carryout the dynamic analysis of bridge pier, time step size required for stability of explicit time integration scheme is required to be deliberated.

5.4.1 Selection of Time Step Size for Analysis

For stability of explicit time integration scheme, maximum permissible time step Δt has been estimated using Eqn. 2.9. In chapter-4, the load histories for various spans have been obtained for maximum train position increment of 0.20/0.25/0.50 m, as mentioned in Table 4.2. From Table 5.3 to Table 5.7, it is noted that minimum time period of system for corresponding span configurations are 0.51 seconds (for 4 m span, Table 5.3), 0.80 seconds (for 14 m span, Table 5.3) and 0.58 seconds (for 30 m span, Table 5.5) respectively. Accordingly, maximum permissible time step size for these spans works out as 0.16 sec, 0.25 sec and 0.18 sec respectively. At lowest speed considered (5 m/s), the time step sizes are 0.04/0.05/0.10 seconds which are less than maximum permissible time step sizes as worked out above. At higher speeds, the time step will be further less thus further improving the stability of solution.

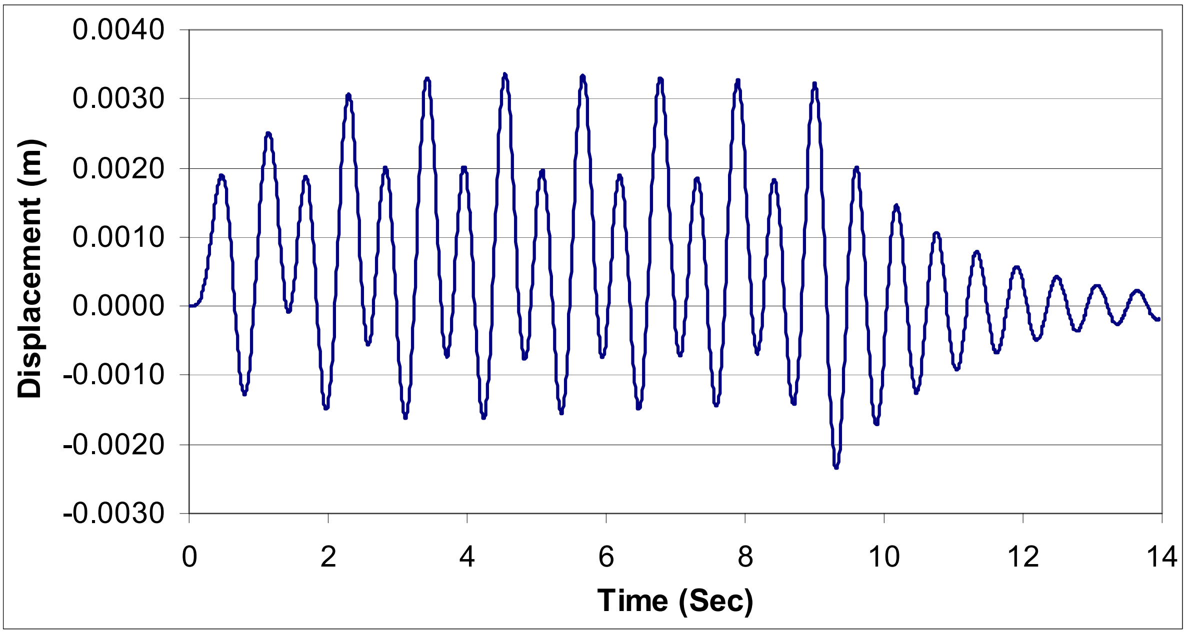
Now all the parameters, data and procedures required for dynamic analysis have been discussed. For carrying out the dynamic analysis, an Excel Spreadsheet has been developed. The sheet has been tested for classical tests, viz., free vibration test under initial displacement and forced vibration test. The results have been compared with standard solutions and found to be satisfactory.

5.4.2 Dynamic Analysis of Bridge Pier

Using, mass M, damping C, stiffness K and load history, response of pier for various span configurations as given in Table 5.1 has been obtained using Eqn. 2.6 for various speeds. Primarily the analysis has been carried out for speed range from 5 m/s to 40 m/s increasing at an increment of 2.5 m/s. However, near the resonance peaks, additional speeds have also been considered. Fig. 5.4 to Fig. 5.10 presents the pier response at train speed of 20 m/s for span of 6 m, 10 m, 14 m, 18 m, 22 m, 26 m and 30 m respectively for bridge parameters of Group-1 (Table 5.3). Fig. 5.11 to Fig. 5.17 presents the pier response at train speed of 20 m/s for span of 32 m, 36 m, 40 m, 45 m, 50 m, 55 m and 60 m respectively for bridge parameters of Group-4 (Table 5.4). Maximum amplitude, Dynamic & static base shear and dynamic amplification factor for each case have also been mentioned below each figure, which has also been summarised in Table 5.8 given below.

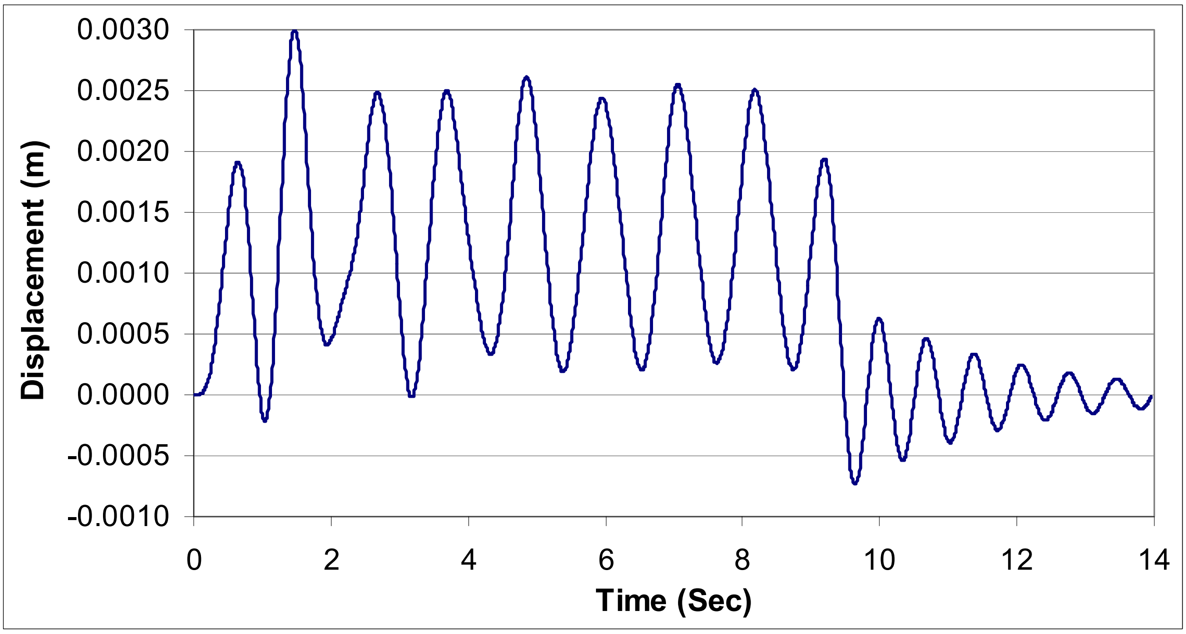
Table 5.8 Response of Bridge Pier with various Spans at 20 m/s Train Speed

S.N.	Span (m)	Max Pier Disp (mm)	Dyn. Base Shear (kN)	Static Base Shear (kN)	DAF	Group
1	6	3.3	85.8	33.1	2.59	1
2	10	3.0	73.3	47.2	1.55	1
3	14	3.0	70.0	54.9	1.28	1
4	18	3.0	69.1	61.6	1.12	1
5	22	3.3	74.6	71.9	1.04	1
6	26	4.3	95.9	86.0	1.11	1
7	30	6.0	132.3	100.6	1.31	1
8	32	1.9	113.9	108.1	1.05	4
9	36	2.1	126.9	120.5	1.05	4
10	40	2.3	137.4	131.3	1.05	4
11	45	2.5	151.9	146.2	1.04	4
12	50	2.9	172.5	163.6	1.05	4
13	55	3.3	195.2	181.1	1.08	4
14	60	3.5	207.2	196.4	1.06	4



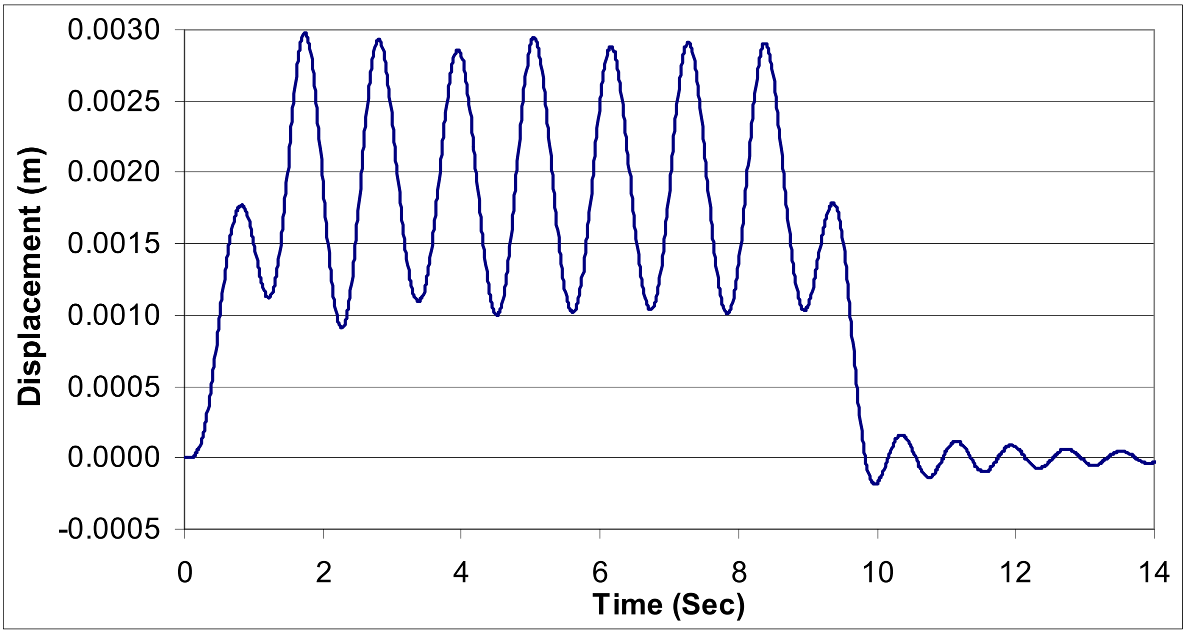
Maximum displacement	**= 0.0033 m**	**Base Shear Dynamic**	**= 85.8 kN**
Base Static	**= 33.1 kN**	**DAF**	**= 2.59**

Fig. 5.4 Pier Response for 6 m Span at Train Speed of 20 m/s (Group–1)



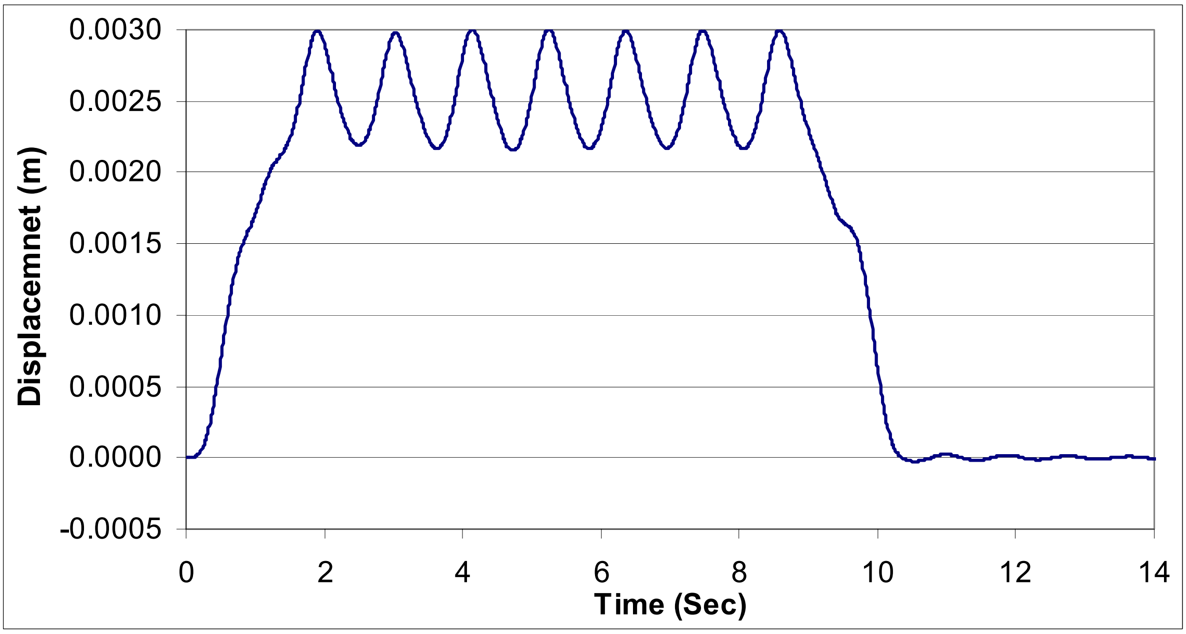
Maximum displacement	**= 0.0030 m**	**Base Shear Dynamic**	**= 73.3 kN**
Base Static	**= 47.2 kN**	**DAF**	**= 1.55**

Fig. 5.5 Pier Response for 10 m Span at Train Speed of 20 m/s (Group–1)



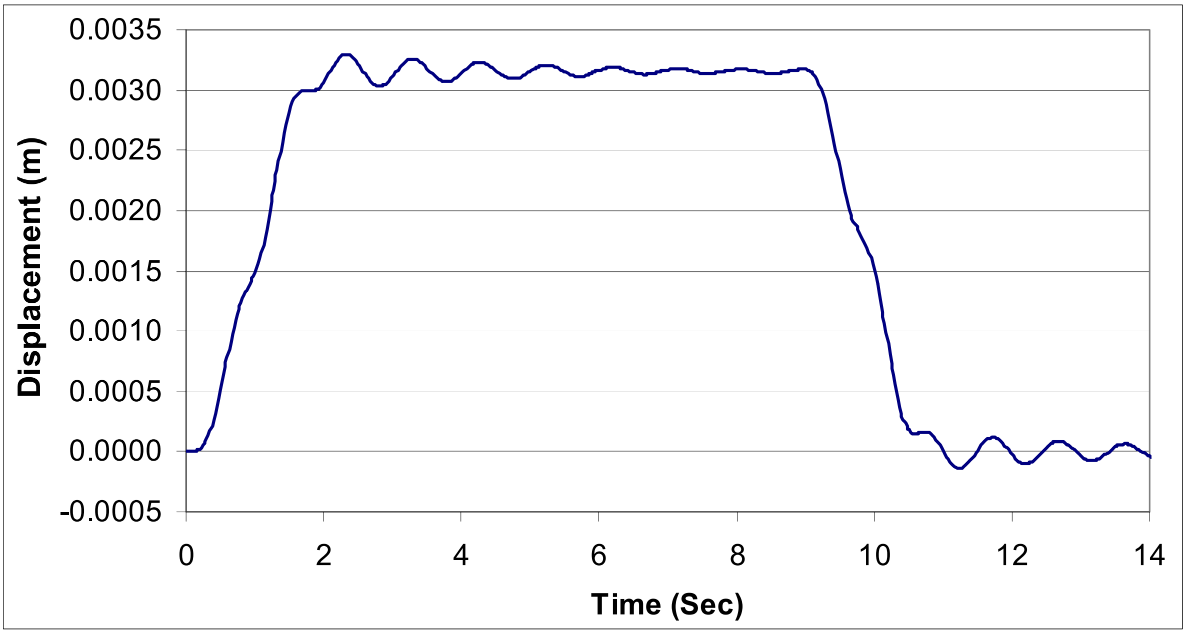
Maximum displacement	**= 0.0030 m**	**Base Shear Dynamic**	**= 70.0 kN**
Base Static	**= 54.9 kN**	**DAF**	**= 1.28**

Fig. 5.6 Pier Response for 14 m Span at Train Speed of 20 m/s (Group–1)



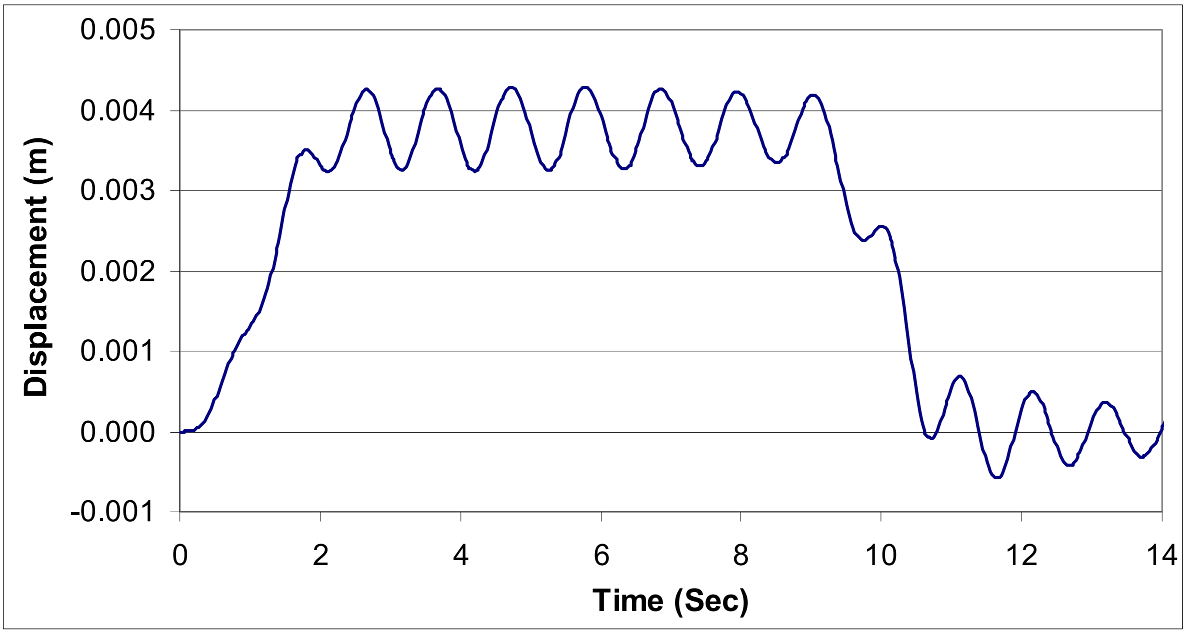
Maximum displacement	**= 0.0030 m**	**Base Shear Dynamic**	**= 69.1 kN**
Base Static	**= 61.6 kN**	**DAF**	**= 1.12**

Fig. 5.7 Pier Response for 18 m Span at Train Speed of 20 m/s (Group–1)



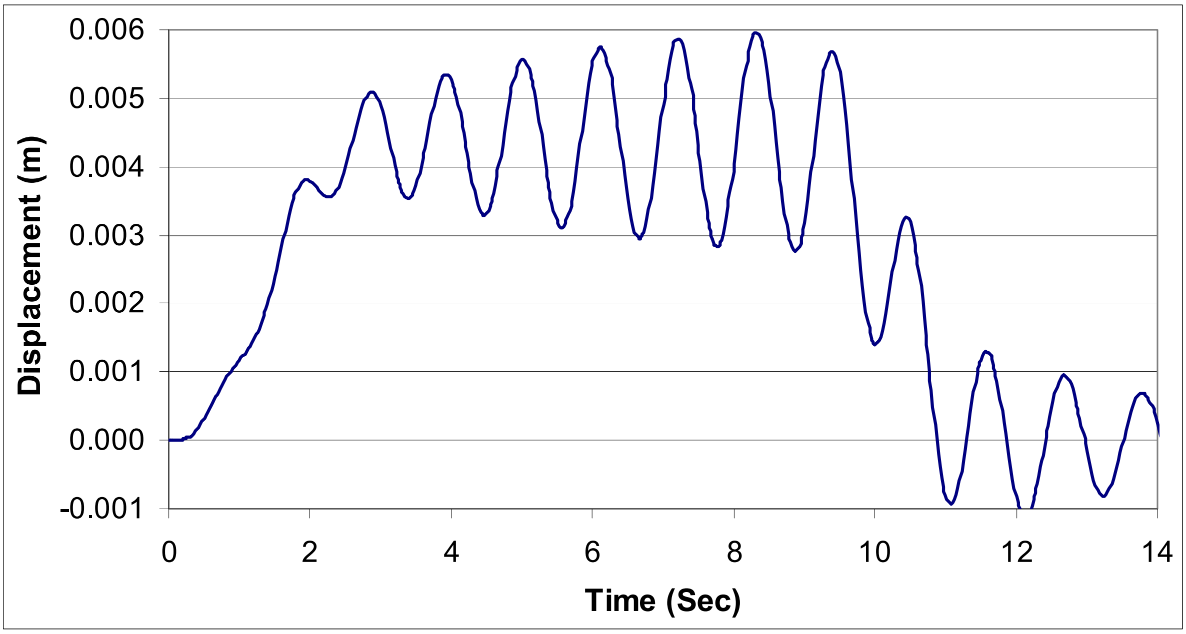
Maximum displacement	**= 0.0033 m**	**Base Shear Dynamic**	**= 74.6 kN**
Base Static	**= 71.9 kN**	**DAF**	**= 1.04**

Fig. 5.8 Pier Response for 22 m Span at Train Speed of 20 m/s (Group–1)



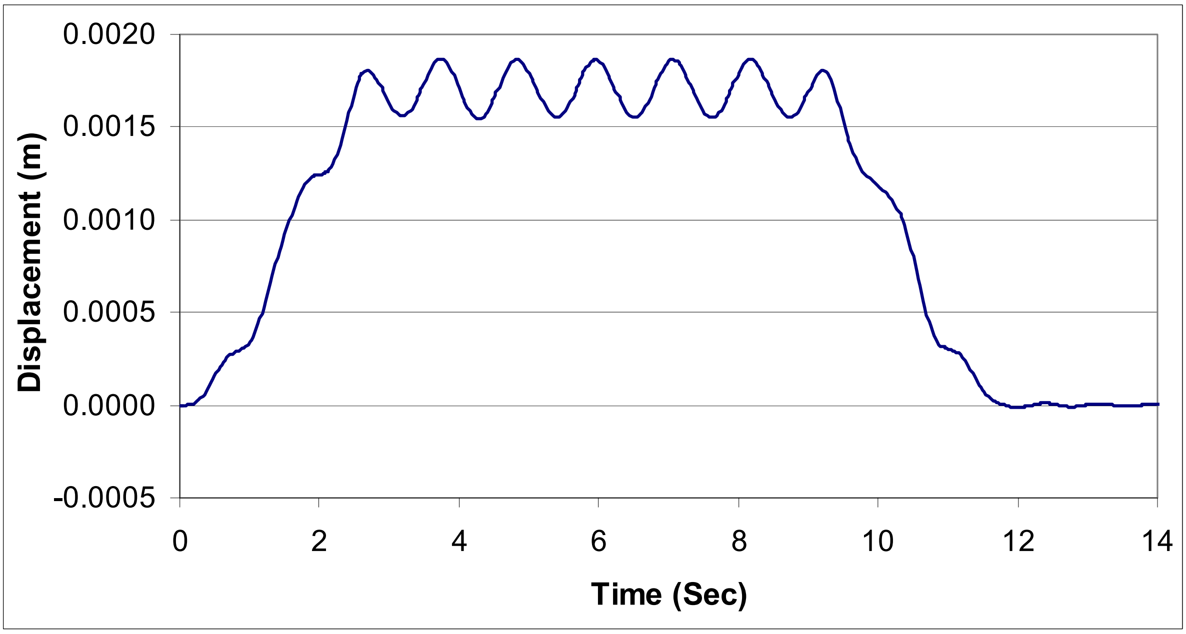
Maximum displacement	**= 0.0043 m**	**Base Shear Dynamic**	**= 95.9 kN**
Base Static	**= 86.0 kN**	**DAF**	**= 1.11**

Fig. 5.9 Pier Response for 26 m Span at Train Speed of 20 m/s (Group–1)



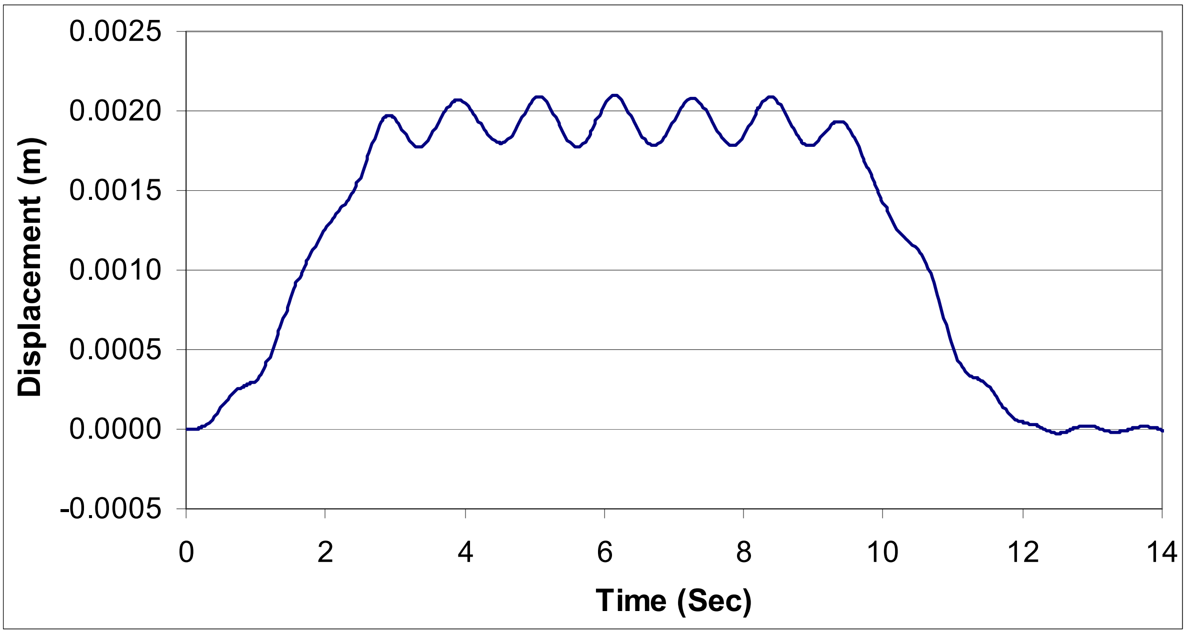
Maximum displacement	**= 0.0060 m**	**Base Shear Dynamic**	**= 132.3 kN**
Base Static	**= 100.60 kN**	**DAF**	**= 1.31**

Fig. 5.10 Pier Response for 30 m Span at Train Speed of 20 m/s (Group–1)



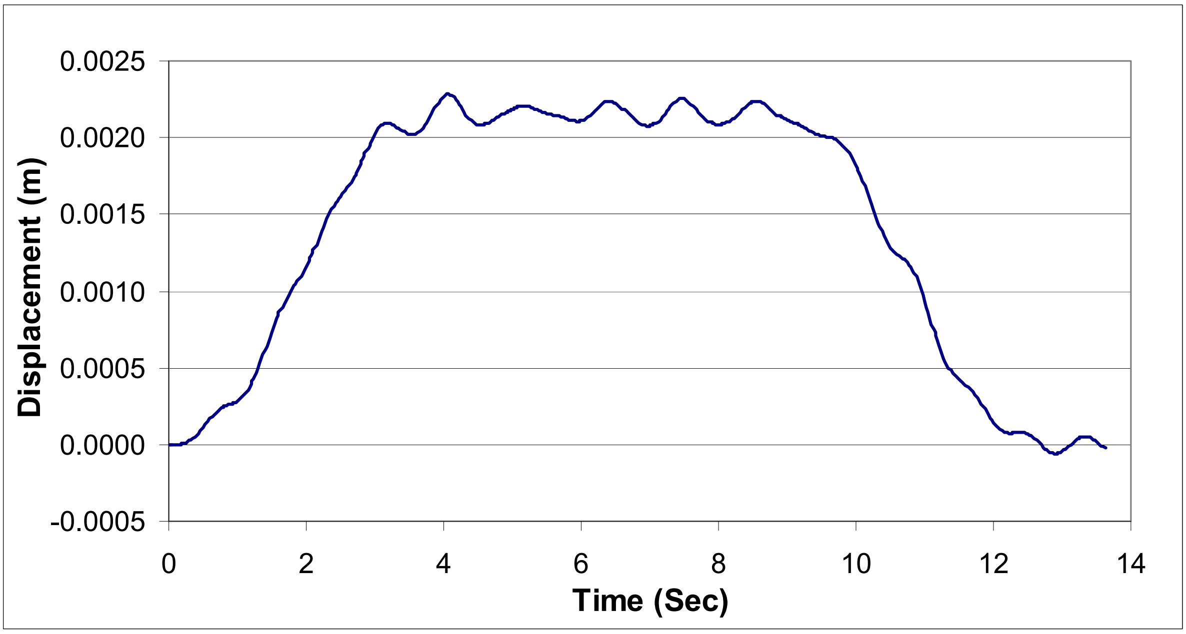
Maximum displacement	**= 0.0019 m**	**Base Shear Dynamic**	**= 113.9 kN**
Base Static	**= 108.1 kN**	**DAF**	**= 1.05**

Fig. 5.11 Pier Response for 32 m Span at Train Speed of 20 m/s (Group–4)



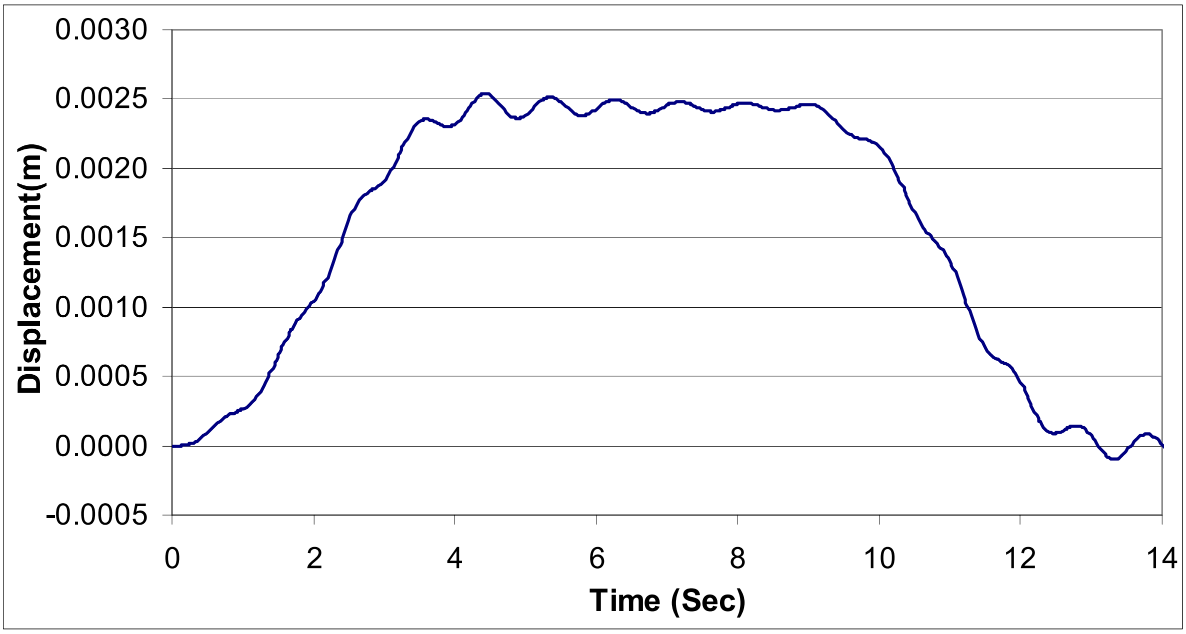
Maximum displacement	**= 0.0021 m**	**Base Shear Dynamic**	**= 126.9 kN**
Base Static	**= 120.5 kN**	**DAF**	**= 1.05**

Fig. 5.12 Pier Response for 36 m Span at Train Speed of 20 m/s (Group–4)



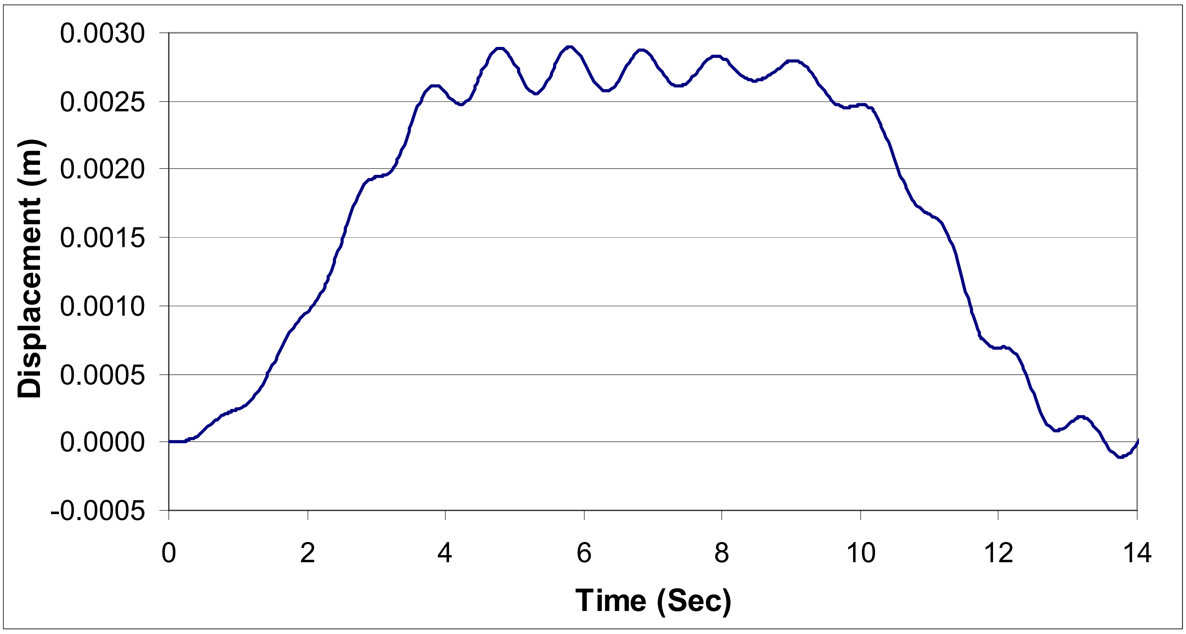
Maximum displacement	**= 0.0023 m**	**Base Shear Dynamic**	**= 137.4 kN**
Base Static	**= 131.3 kN**	**DAF**	**= 1.05**

Fig. 5.13 Pier Response for 40 m Span at Train Speed of 20 m/s (Group–4)



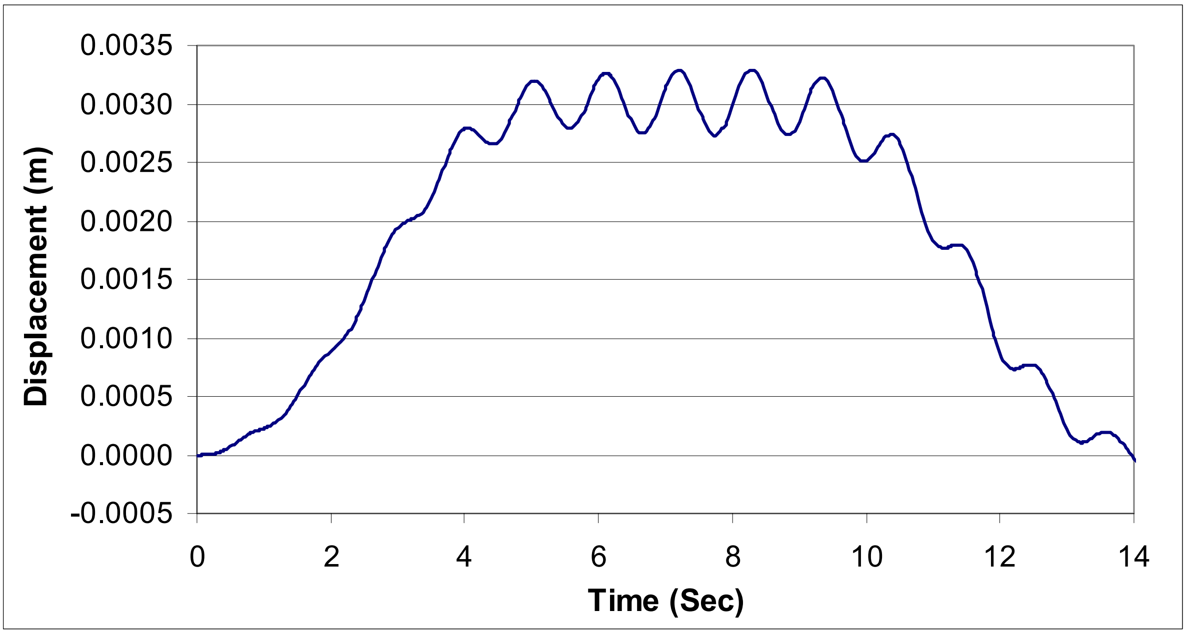
Maximum displacement	**= 0.0025 m**	**Base Shear Dynamic**	**= 151.9 kN**
Base Static	**= 146.2 kN**	**DAF**	**= 1.04**

Fig. 5.14 Pier Response for 45 m Span at Train Speed of 20 m/s (Group–4)



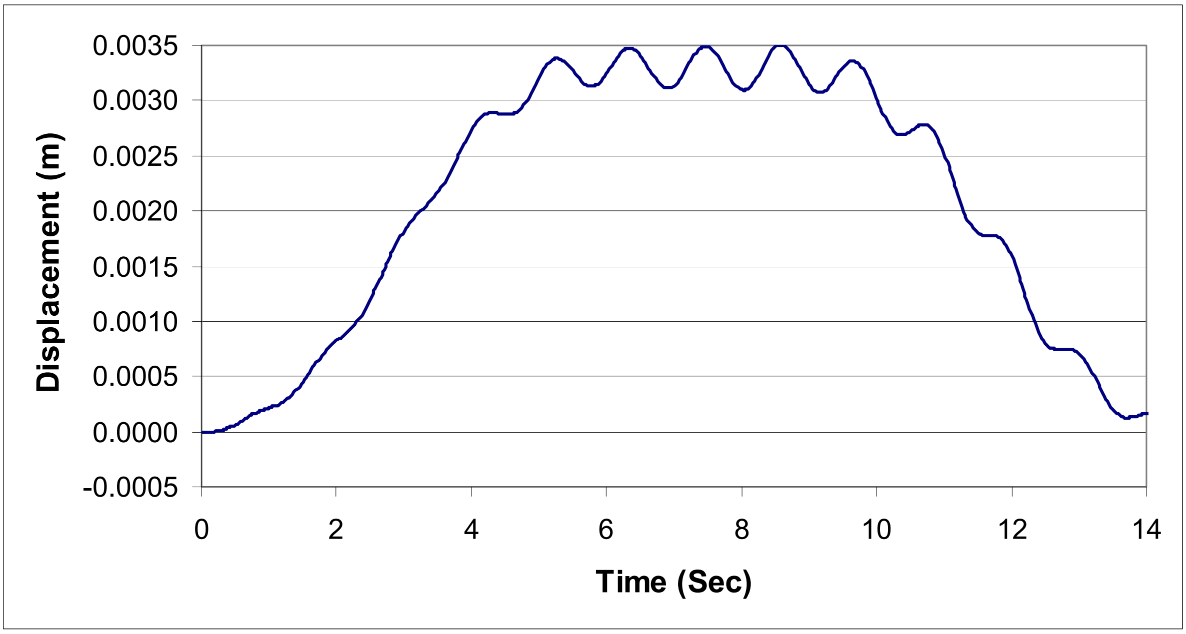
Maximum displacement	**= 0.0029 m**	**Base Shear Dynamic**	**= 172.5 kN**
Base Static	**= 163.6 kN**	**DAF**	**= 1.05**

Fig. 5.15 Pier Response for 50 m Span at Train Speed of 20 m/s (Group–4)



Maximum displacement	**= 0.0033**	**Base Shear Dynamic**	**= 195.2 kN**
Base Static	**= 181.1 kN**	**DAF**	**= 1.08**

Fig. 5.16 Pier Response for 55 m Span at Train Speed of 20 m/s (Group–4)



Maximum displacement	**= 0.0035 m**	**Base Shear Dynamic**	**= 207.2 kN**
Base Static	**= 196.4 kN**	**DAF**	**= 1.06**

Fig. 5.17 Pier Response for 60 m Span at Train Speed of 20 m/s (Group–4)

The data of Table 5.8 has been plotted in Fig 5.18 for maximum pier displacement and in Fig 5.19 for centrifugal force on pier.

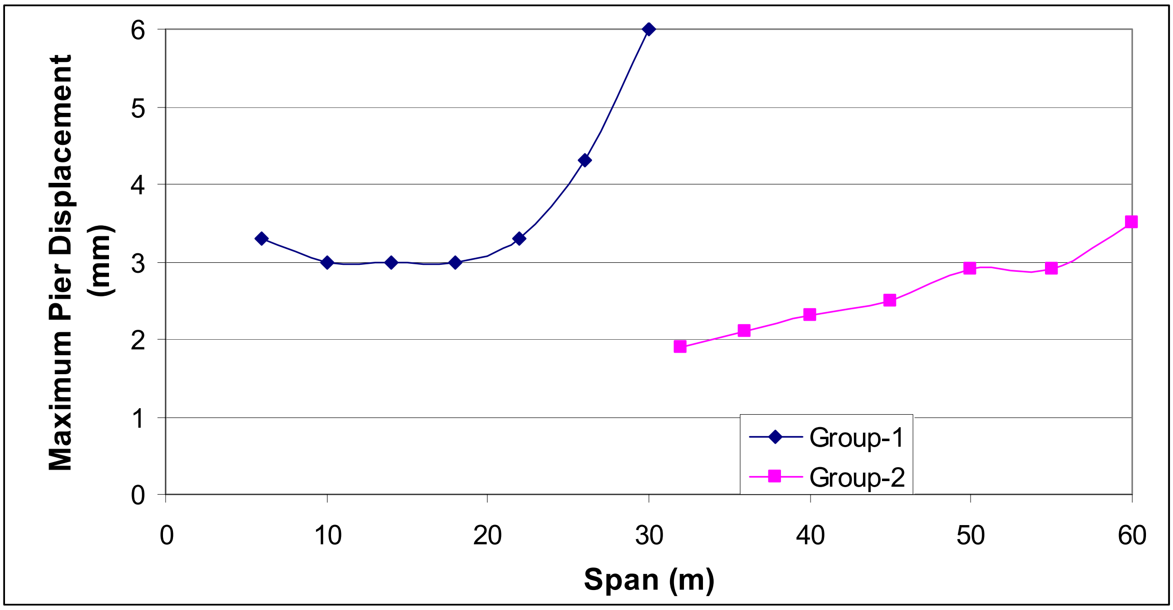


Fig. 5.18 Maximum Pier Displacement at Train Speed of 20 m/s

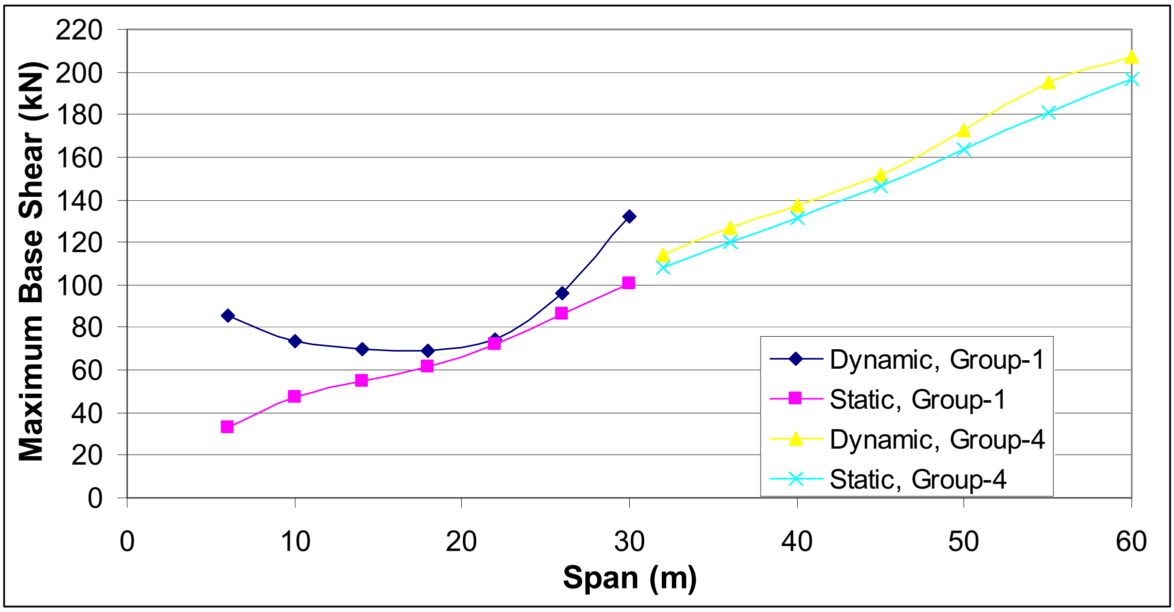


Fig. 5.19 Maximum Pier Base Shear at Train Speed of 20 m/s

From the above, it is noted that maximum pier displacement and maximum dynamic base shear remains almost constant for spans less than one coach length. However for longer spans, these parameters increase with increase in span.

Fig 5.20 presents the variation of dynamic amplification factor with span for train speed of 20 m/s. From the figure, it is noted that dynamic amplification factor for spans less that half coach length, i.e., 11 m is very high (1.55-2.59) even at a speed of 20 m/s. However, it is nearly unity (1.04) for span length equal to one coach length, i.e., 22 m. For span lengths beyond 22m, it further starts increasing and goes upto 1.31 for 30 m span length. For longer spans (32m to 60 m), it again reduces to nearly unity and varies in narrow range from 1.04 to 1.08.

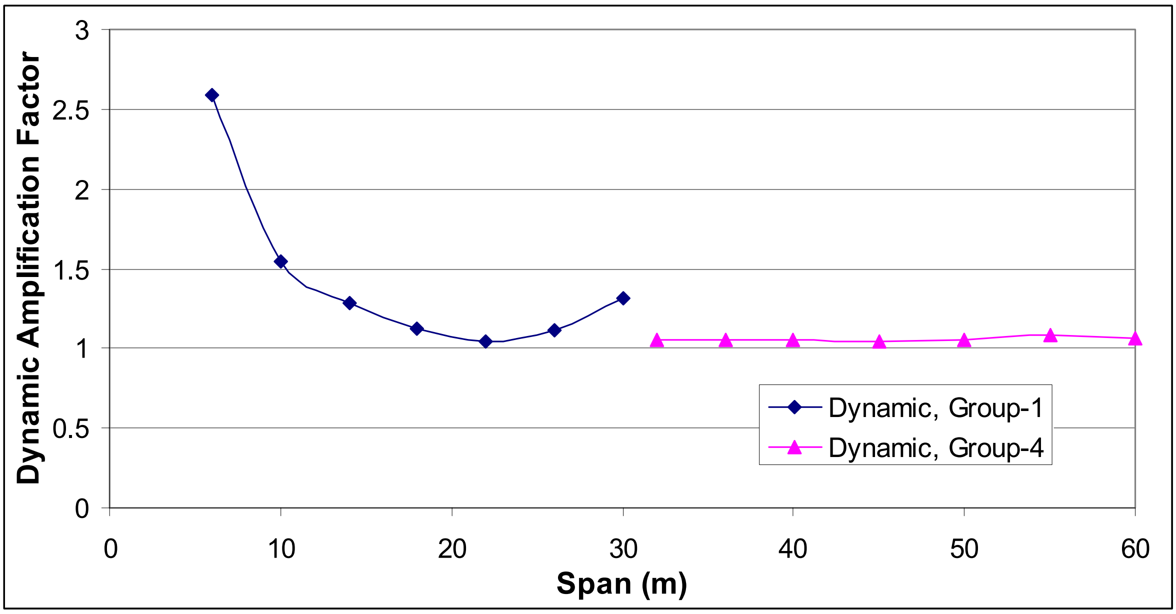
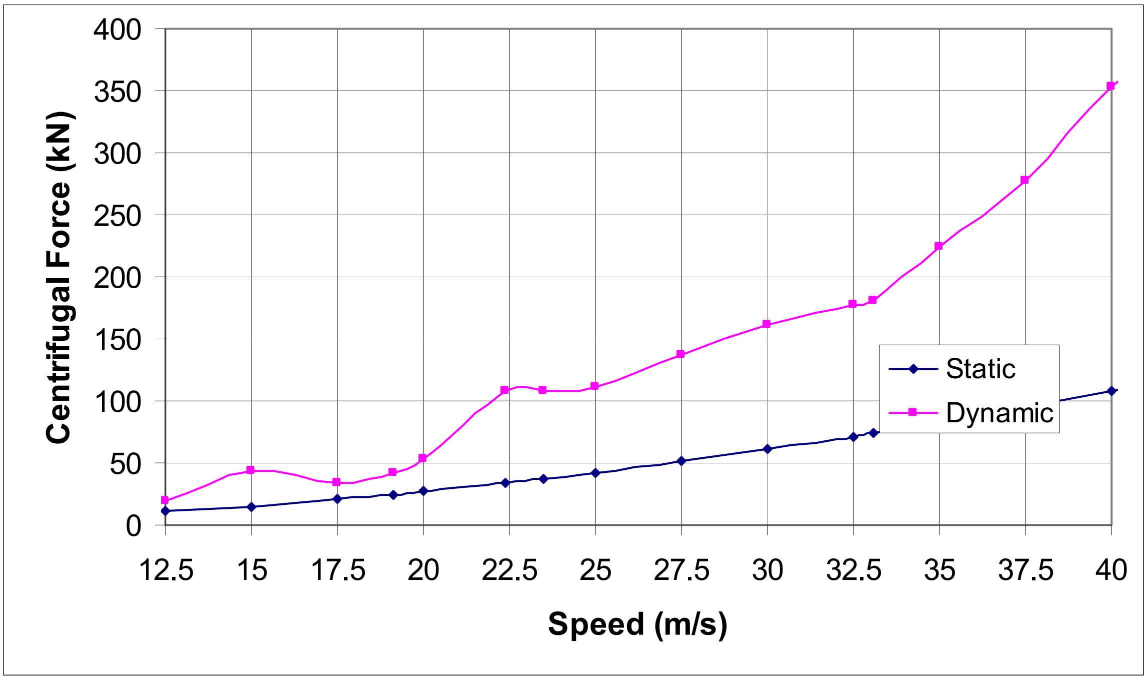


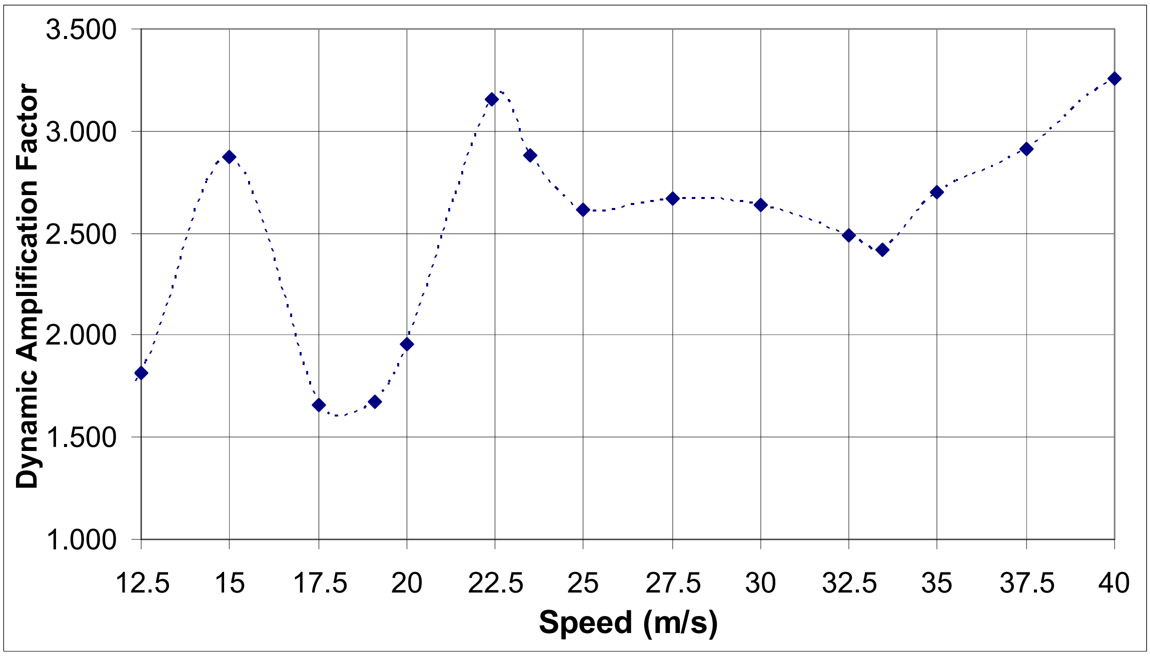
Fig. 5.20 Variation of Dynamic Amplification Factor at Train Speed of 20 m/s

Analysis of the spans for bridge parameters mentioned above (group-1 and group-4) has been carried out at other train speeds (5 to 40 m/s) also. Centrifugal force (both static and dynamic) and DAF for each case has been calculated as presented above. Since, it is maximum static and dynamic base shear along with DAF which is of importance in design; the same have been plotted for Group 1 as a function of span for 4-30 m spans @ 2 m from Fig. 5.21 to 5.48 and for Group 4 for 30 m, 32 m, 36 m, 40-60 m @ 5 m interval from Fig. 5.49 to 5.64. The odd and even number of Figure indicate variation of base shear and DAF respectively.



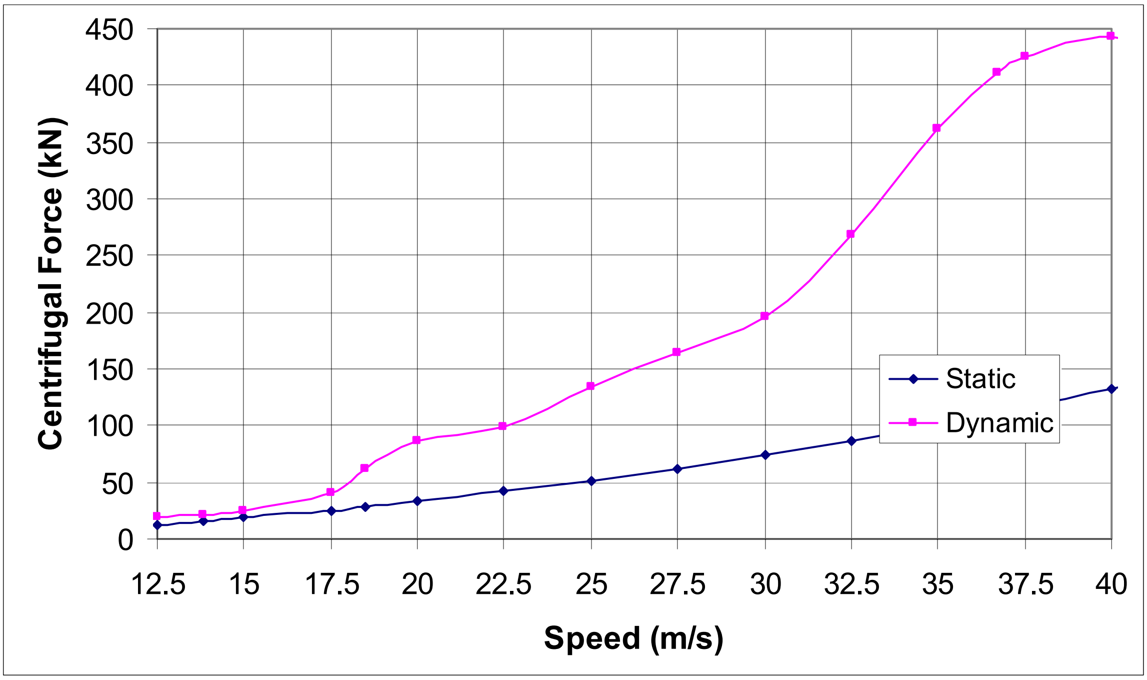
Group-1: Pier Dia = 1.5 m Pier Height = 8.0 m Acs = 2.1 m^2

Fig. 5.21 Centrifugal Force on Pier at Different train speeds on 4 m span



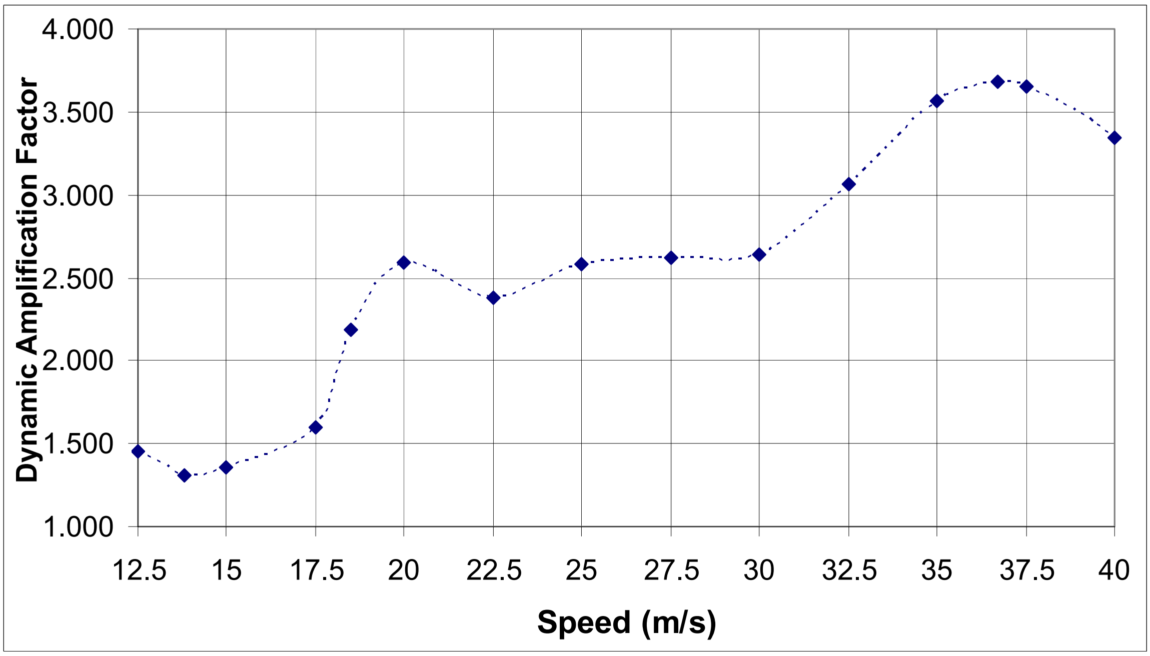
Group-1: Pier Dia = 1.5 m Pier Height = 8.0 m Acs = 2.1 m^2

Fig. 5.22 Dynamic Amplification Factor at Different train speeds for 4 m span



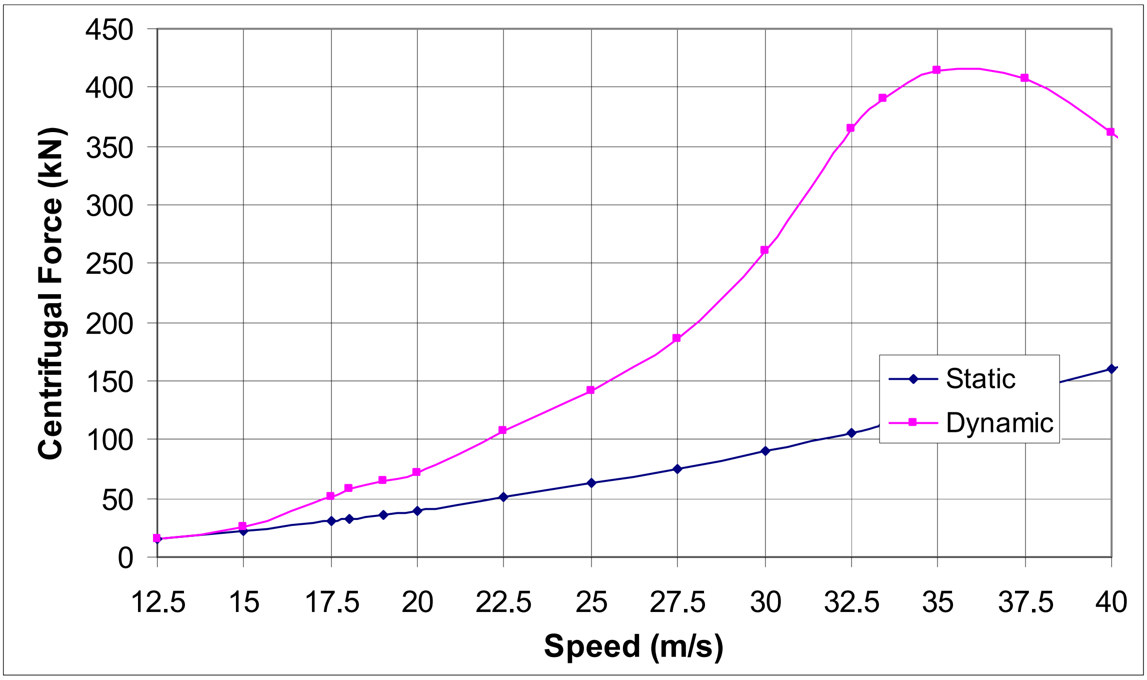
Group-1: Pier Dia = 1.5 m Pier Height = 8.0 m Acs = 2.1 m^2

Fig. 5.23 Centrifugal Force on Pier at Different train speeds on 6 m span



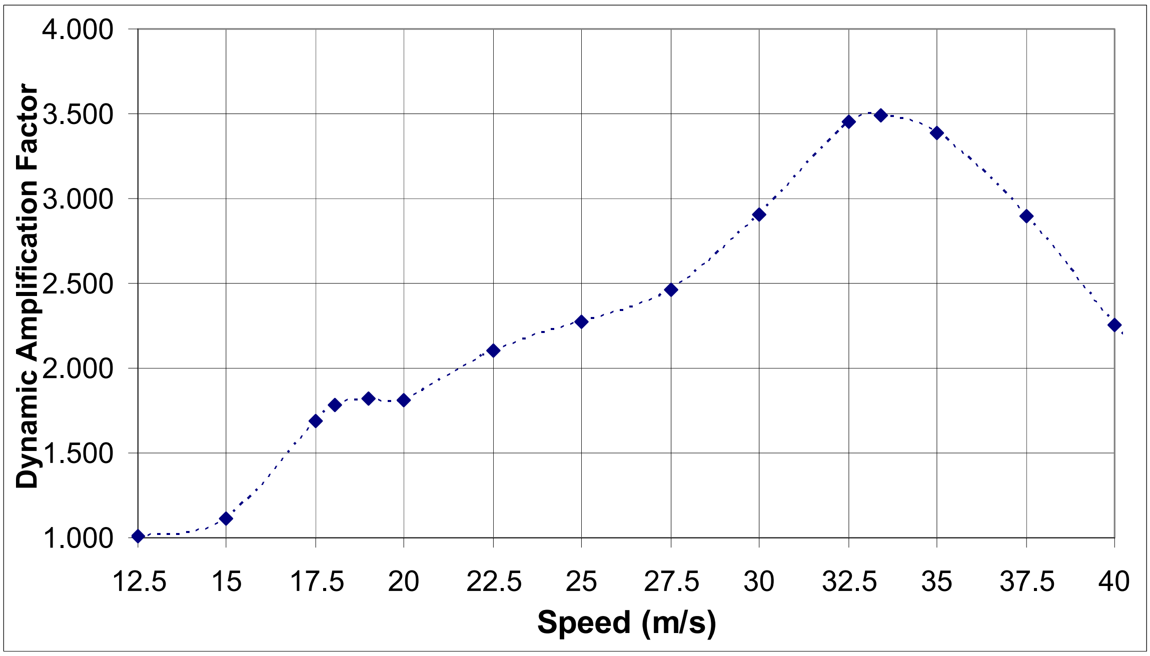
Group-1: Pier Dia = 1.5 m Pier Height = 8.0 m Acs = 2.1 m^2

Fig. 5.24 Dynamic Amplification Factor at Different train speeds for 6 m span



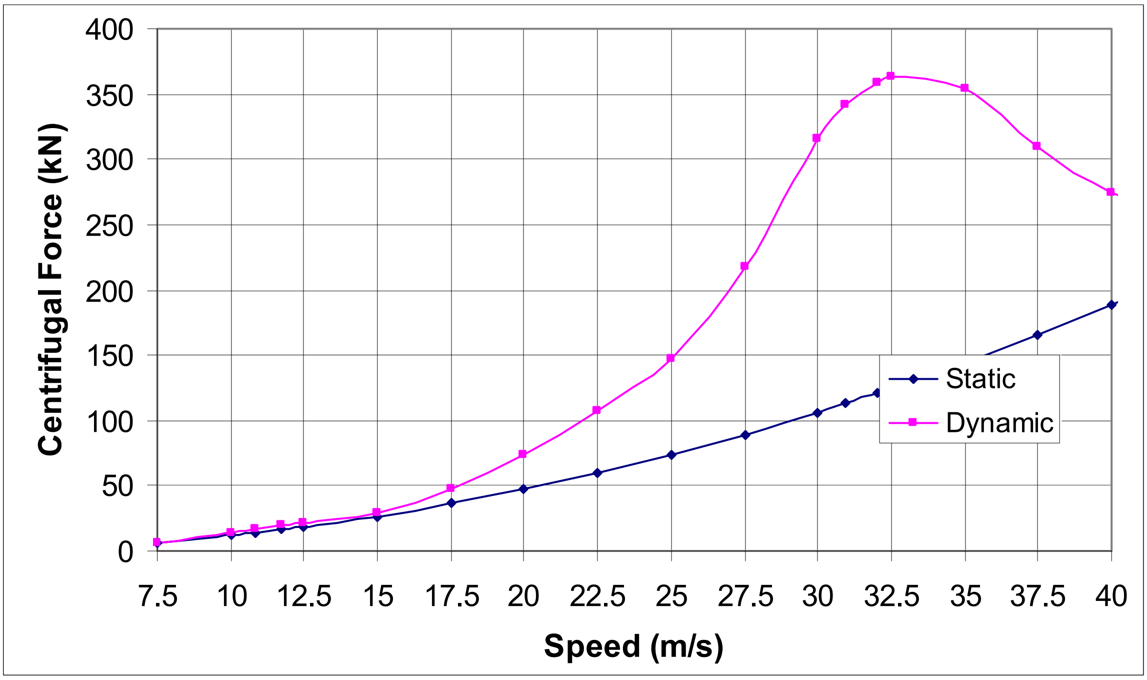
Group-1: Pier Dia = 1.5 m Pier Height = 8.0 m Acs = 2.1 m^2

Fig. 5.25 Centrifugal Force on Pier at Different train speeds on 8 m span



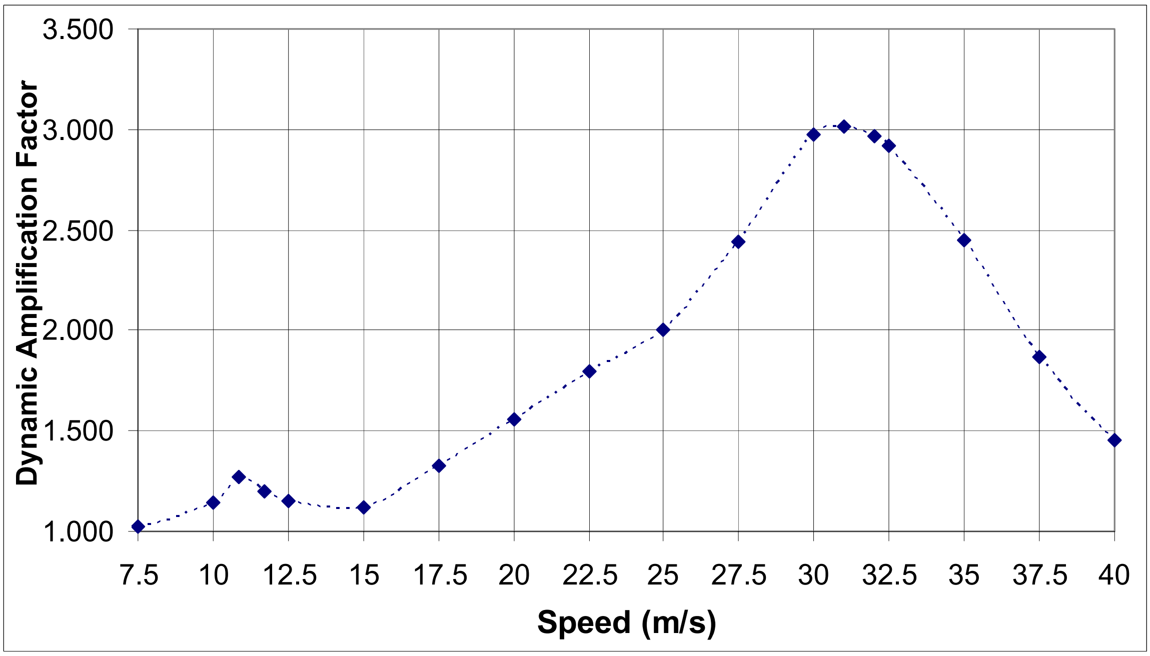
Group-1: Pier Dia = 1.5 m Pier Height = 8.0 m Acs = 2.1 m^2

Fig. 5.26 Dynamic Amplification Factor at Different train speeds for 8 m span



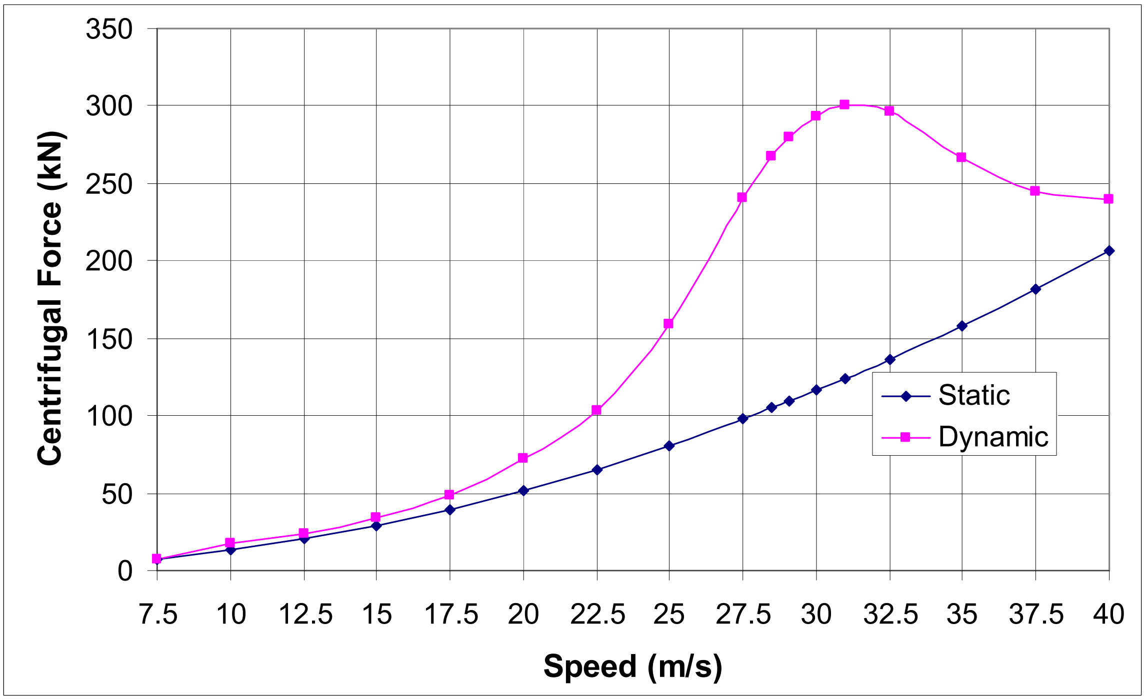
Group-1: Pier Dia = 1.5 m Pier Height = 8.0 m Acs = 2.1 m^2

Fig. 5.27 Centrifugal Force on Pier at Different train speeds on 10 m span



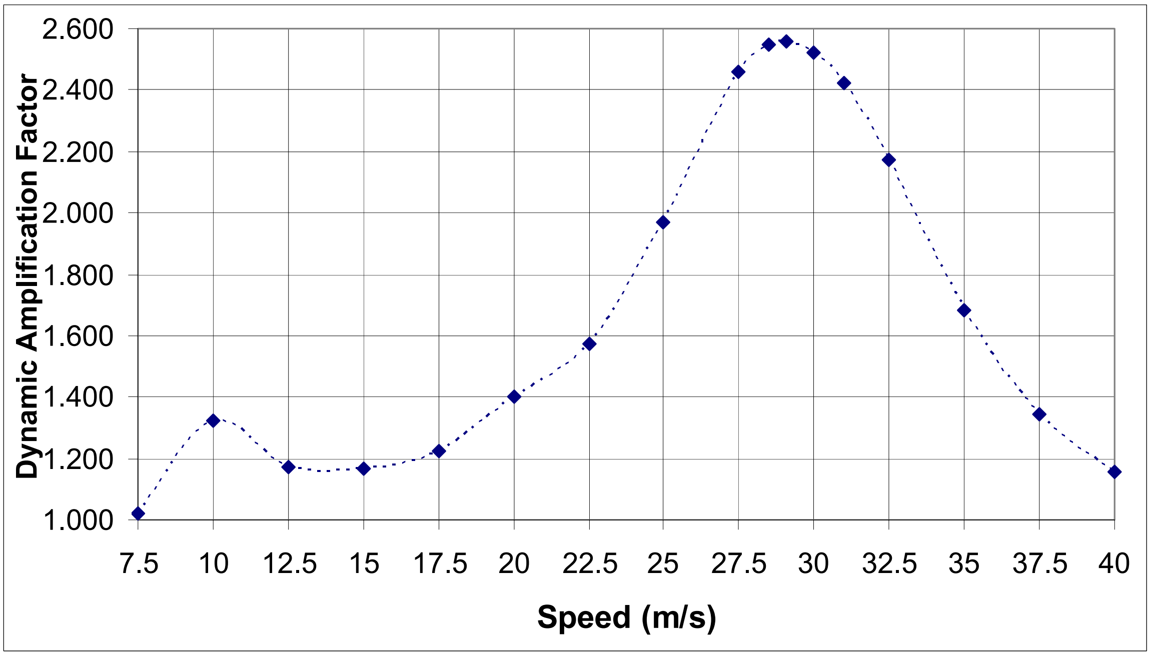
Group-1: Pier Dia = 1.5 m Pier Height = 8.0 m Acs = 2.1 m^2

Fig. 5.28 Dynamic Amplification Factor at Different train speeds for 10 m span



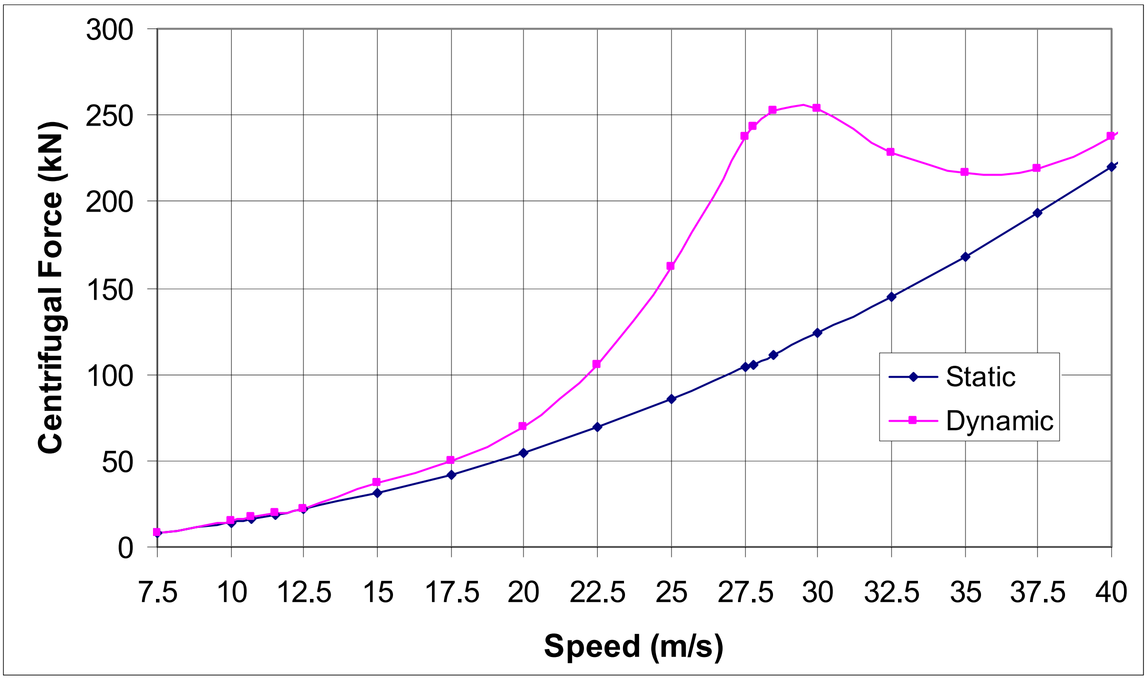
Group-1: Pier Dia = 1.5 m Pier Height = 8.0 m Acs = 2.1 m^2

Fig. 5.29 Centrifugal Force on Pier at Different train speeds on 12 m span



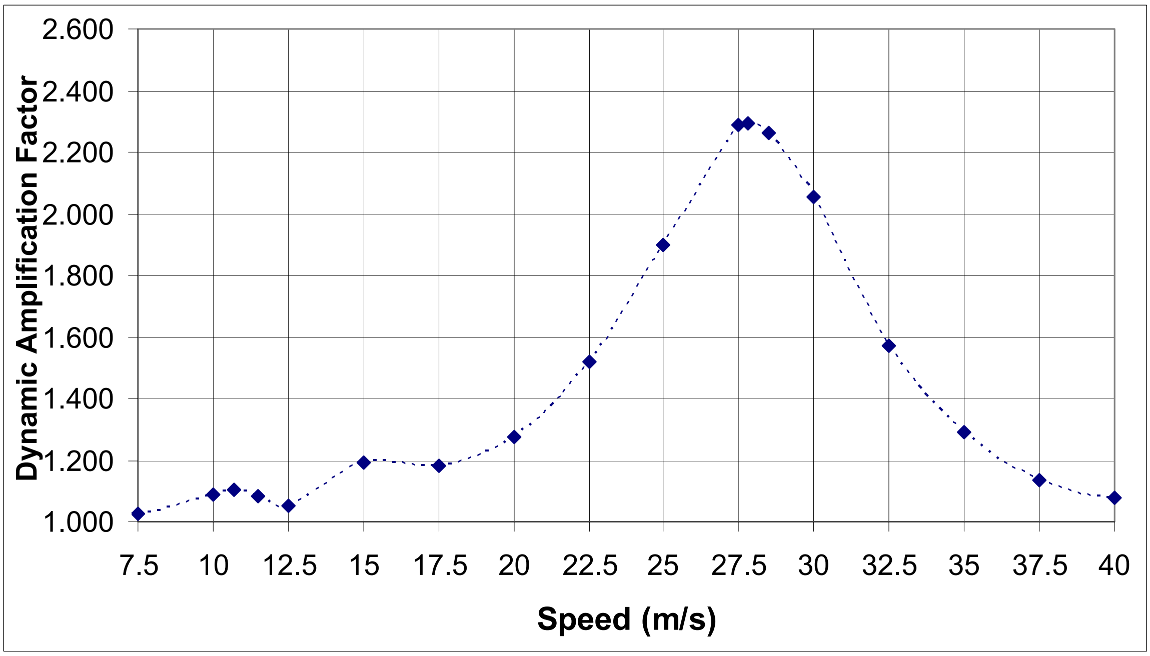
Group-1: Pier Dia = 1.5 m Pier Height = 8.0 m Acs = 2.1 m^2

Fig. 5.30 Dynamic Amplification Factor at Different train speeds for 12 m span



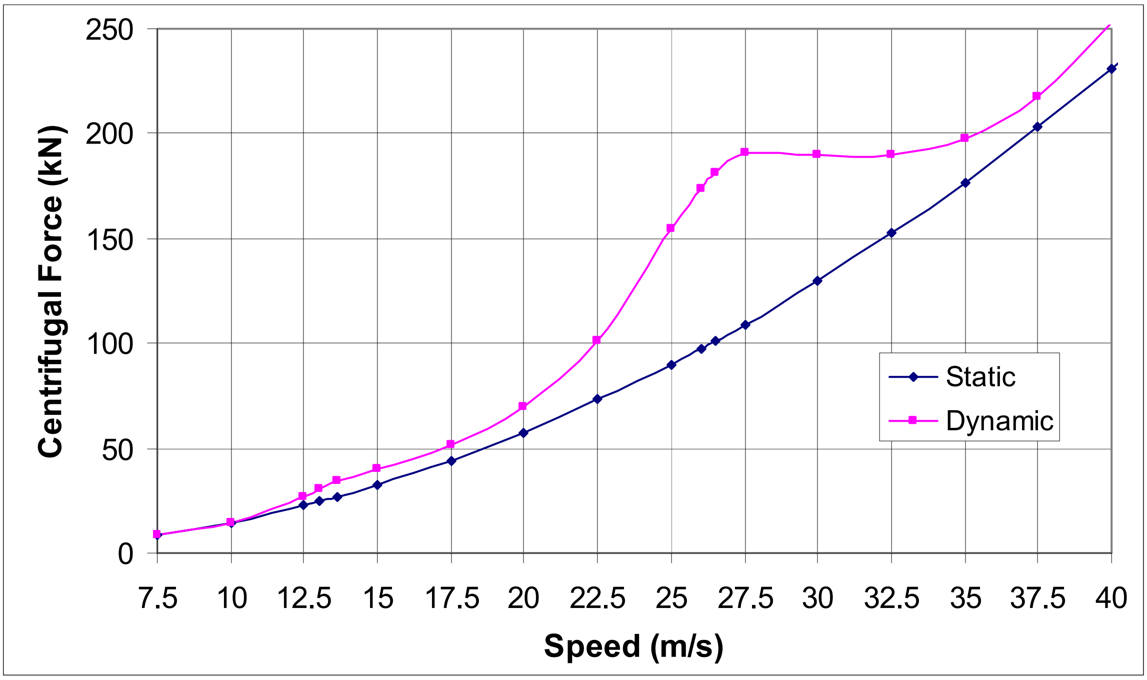
Group-1: Pier Dia = 1.5 m Pier Height = 8.0 m Acs = 2.1 m^2

Fig. 5.31 Centrifugal Force on Pier at Different train speeds on 14 m span



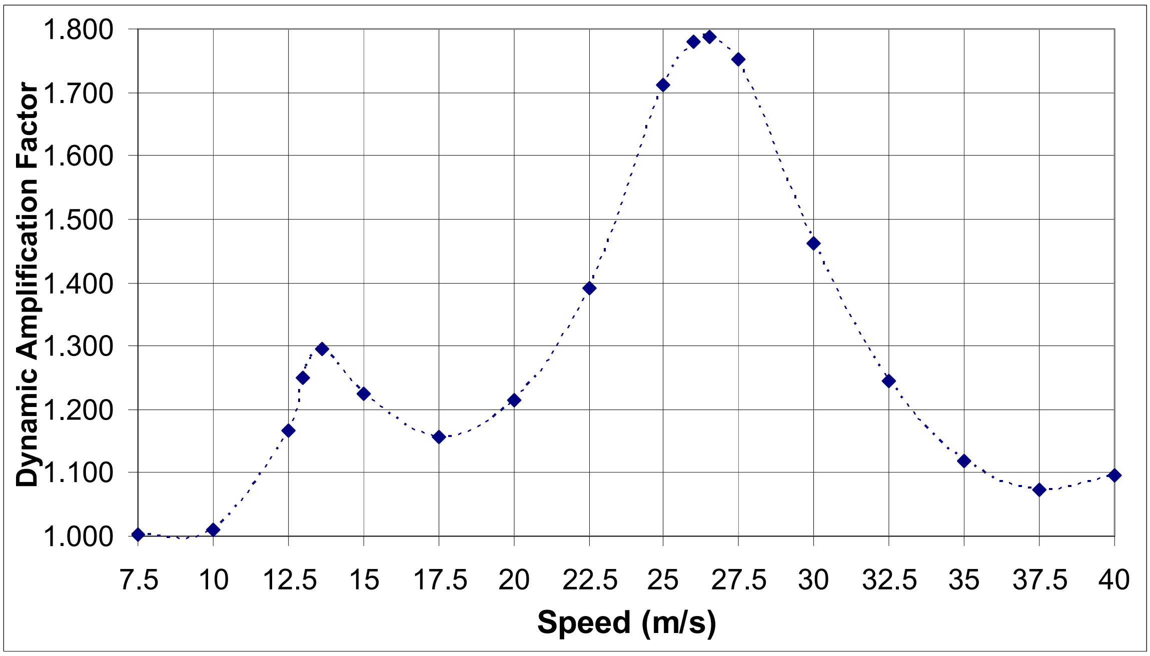
Group-1: Pier Dia = 1.5 m Pier Height = 8.0 m Acs = 2.1 m^2

Fig. 5.32 Dynamic Amplification Factor at Different train speeds for 14 m span



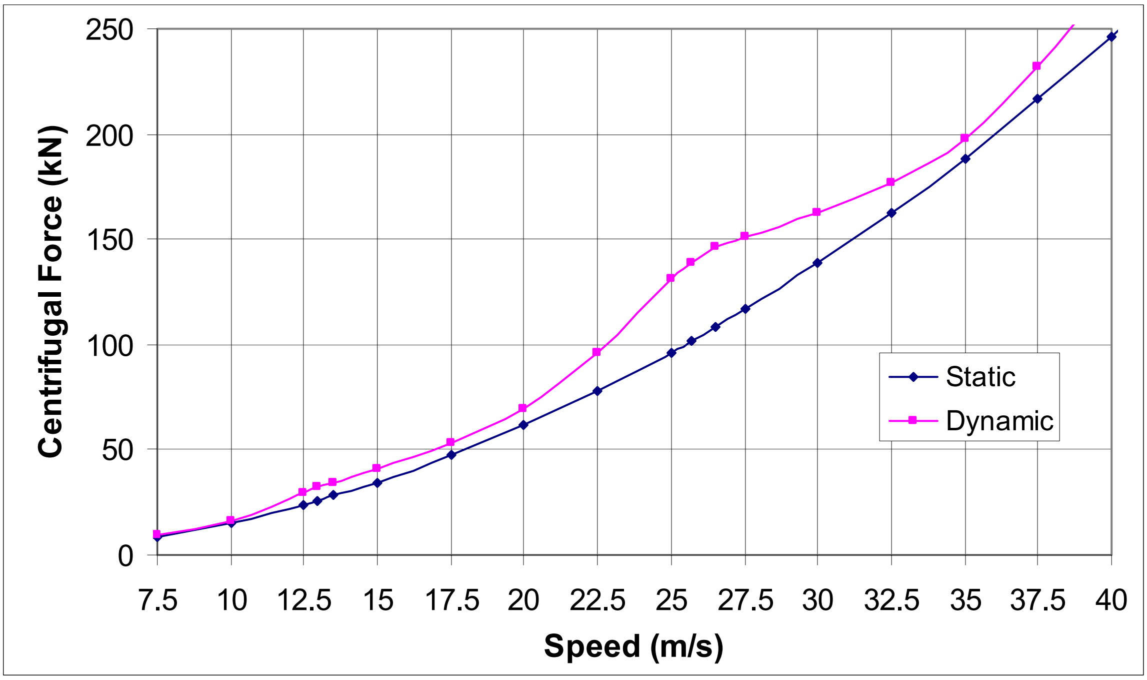
Group-1: Pier Dia = 1.5 m Pier Height = 8.0 m Acs = 2.1 m^2

Fig. 5.33 Centrifugal Force on Pier at Different train speeds on 16 m span



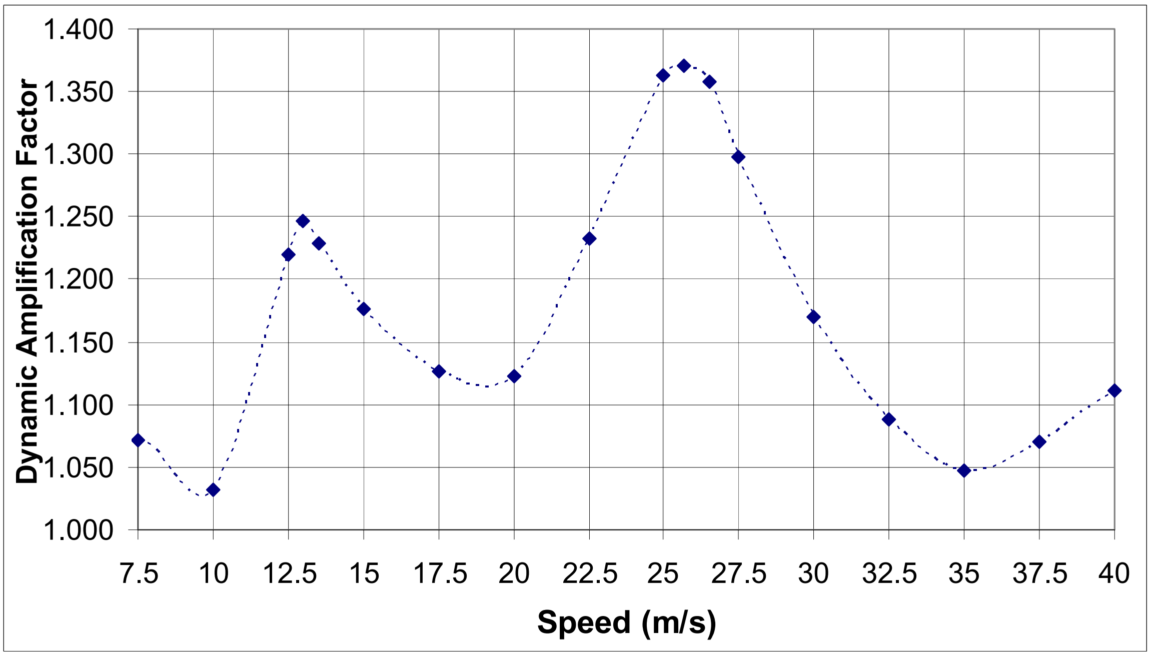
Group-1: Pier Dia = 1.5 m Pier Height = 8.0 m Acs = 2.1 m^2

Fig. 5.34 Dynamic Amplification Factor at Different train speeds for 16 m span



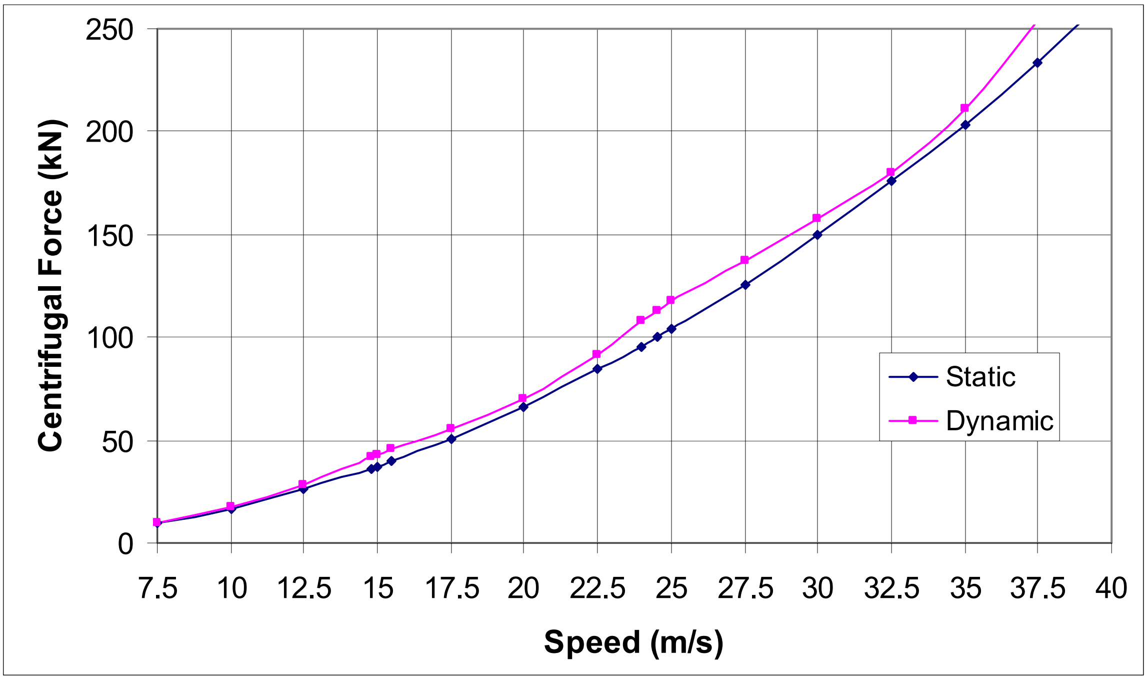
Group-1: Pier Dia = 1.5 m Pier Height = 8.0 m Acs = 2.1 m^2

Fig. 5.35 Centrifugal Force on Pier at Different train speeds on 18 m span



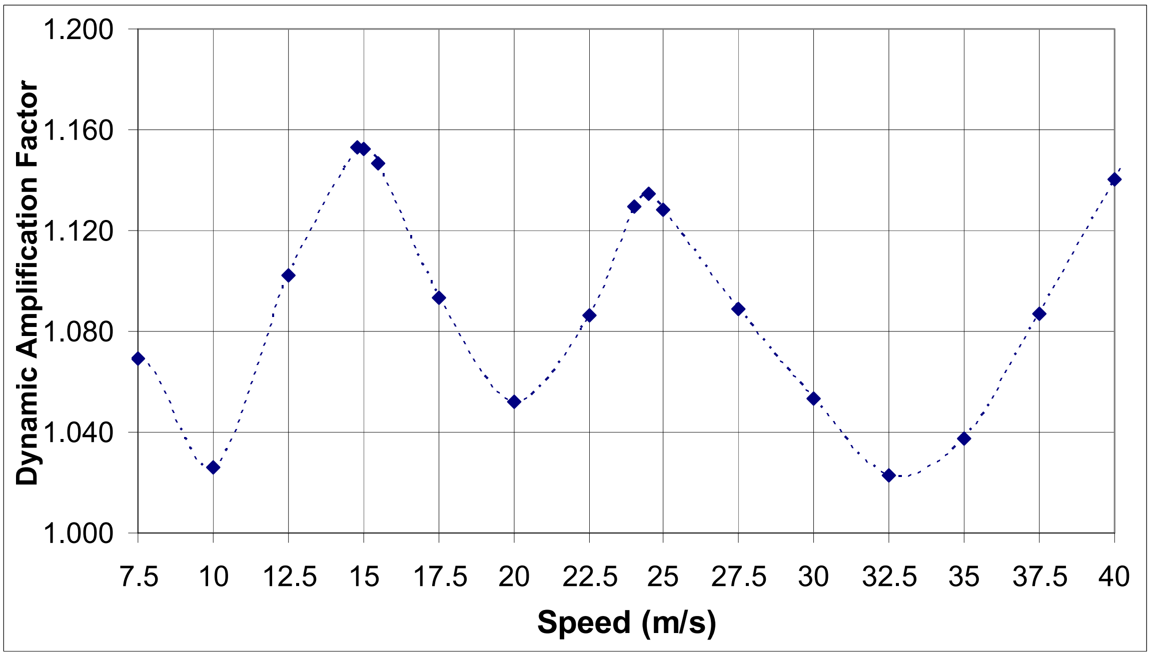
Group-1: Pier Dia = 1.5 m Pier Height = 8.0 m Acs = 2.1 m^2

Fig. 5.36 Dynamic Amplification Factor at Different train speeds for 18 m span



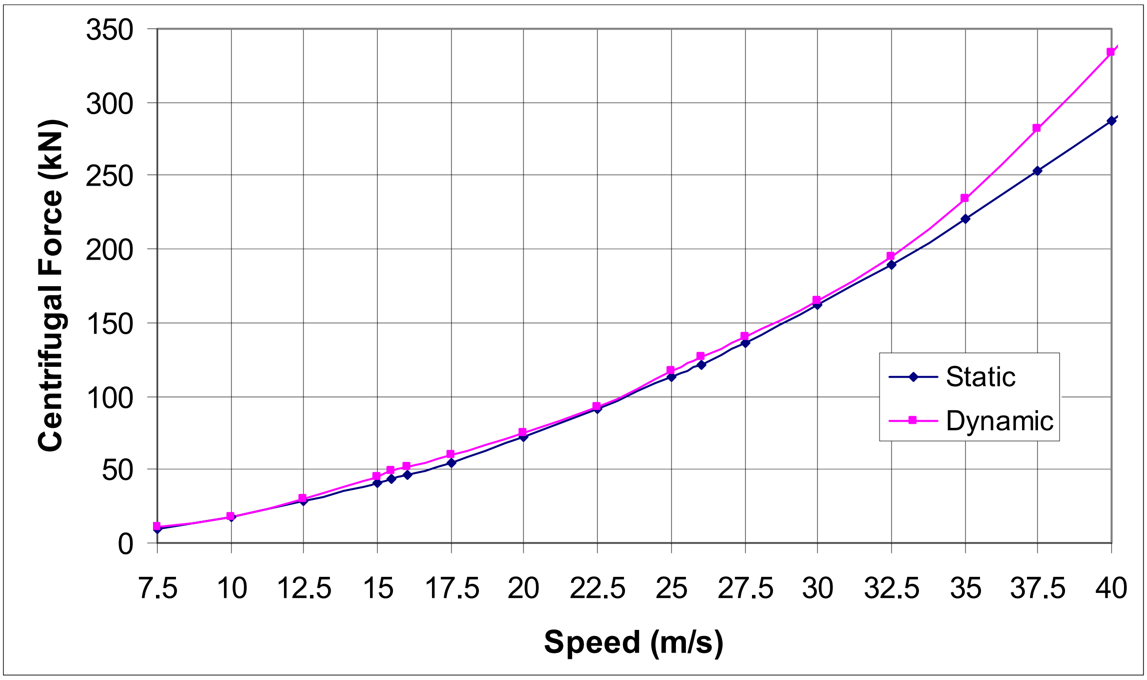
Group-1: Pier Dia = 1.5 m Pier Height = 8.0 m Acs = 2.1 m^2

Fig. 5.37 Centrifugal Force on Pier at Different train speeds on 20 m span



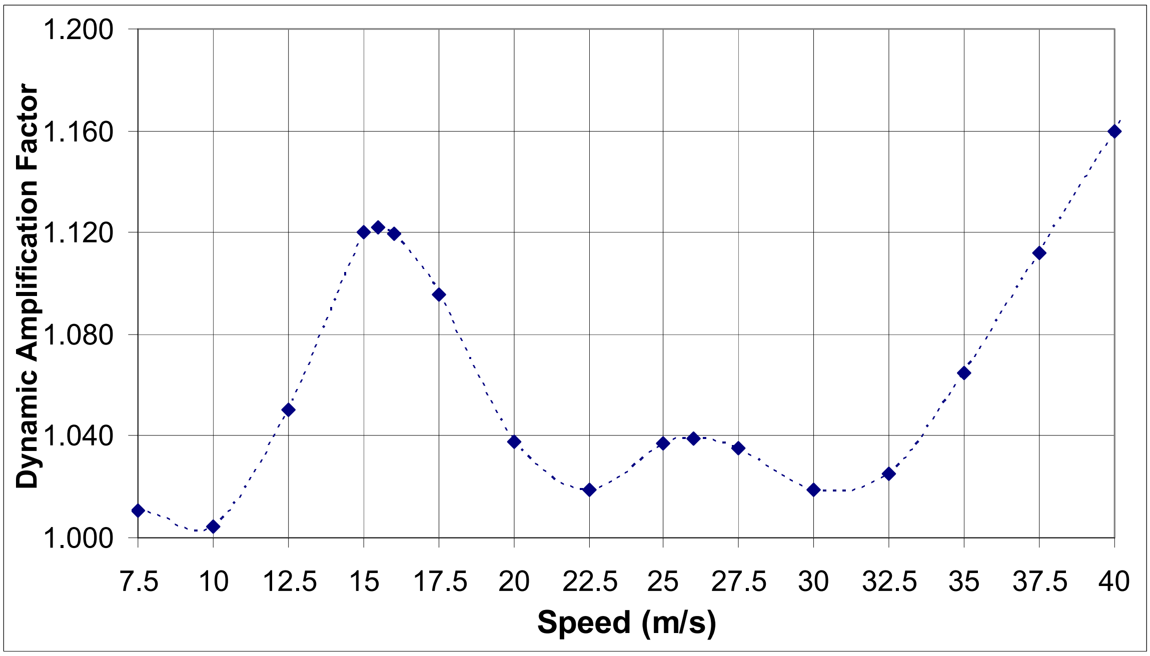
Group-1: Pier Dia = 1.5 m Pier Height = 8.0 m Acs = 2.1 m^2

Fig. 5.38 Dynamic Amplification Factor at Different train speeds for 20 m span



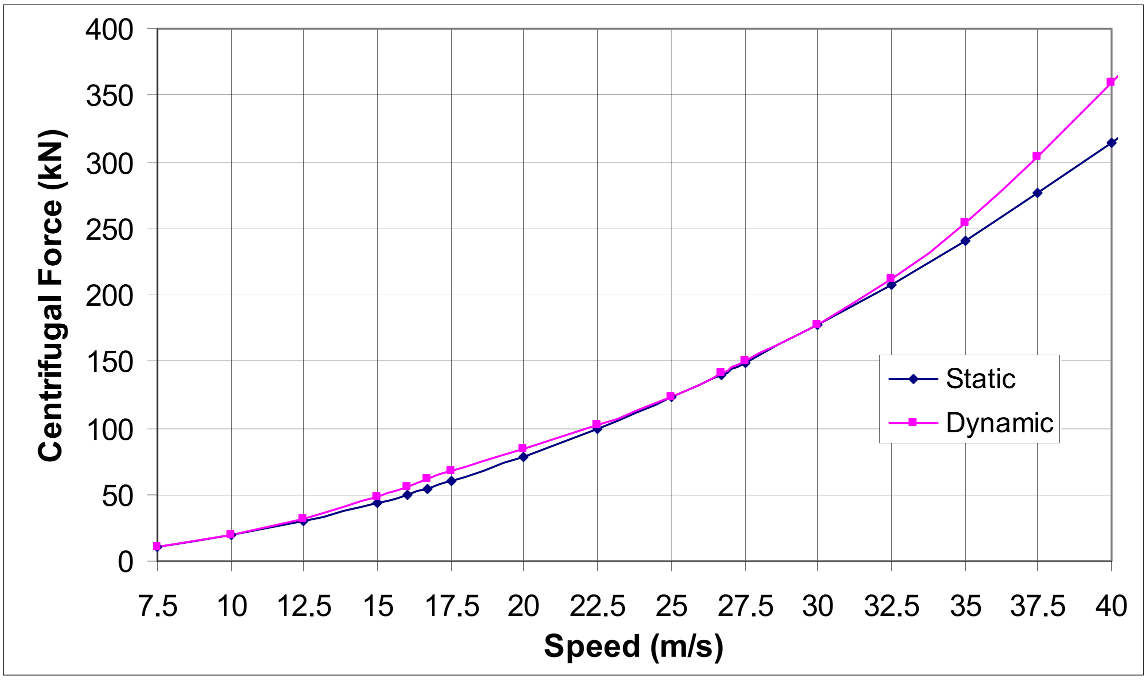
Group-1: Pier Dia = 1.5 m Pier Height = 8.0 m Acs = 2.1 m^2

Fig. 5.39 Centrifugal Force on Pier at Different train speeds on 22 m span



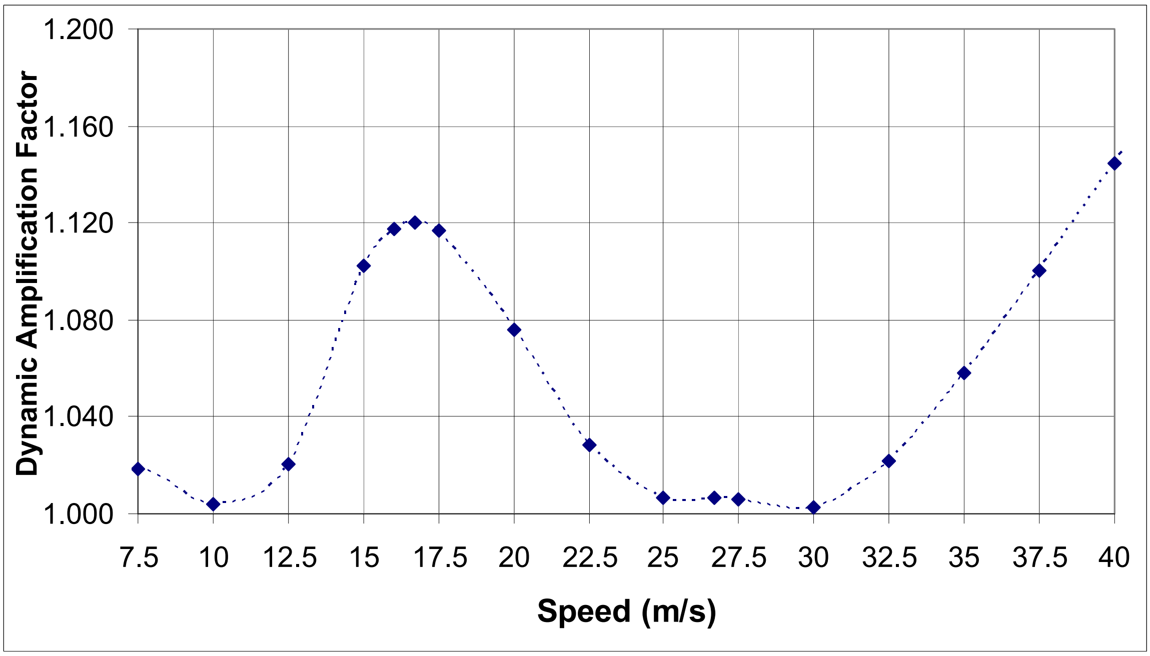
Group-1: Pier Dia = 1.5 m Pier Height = 8.0 m Acs = 2.1 m^2

Fig. 5.40 Dynamic Amplification Factor at Different train speeds for 22 m span



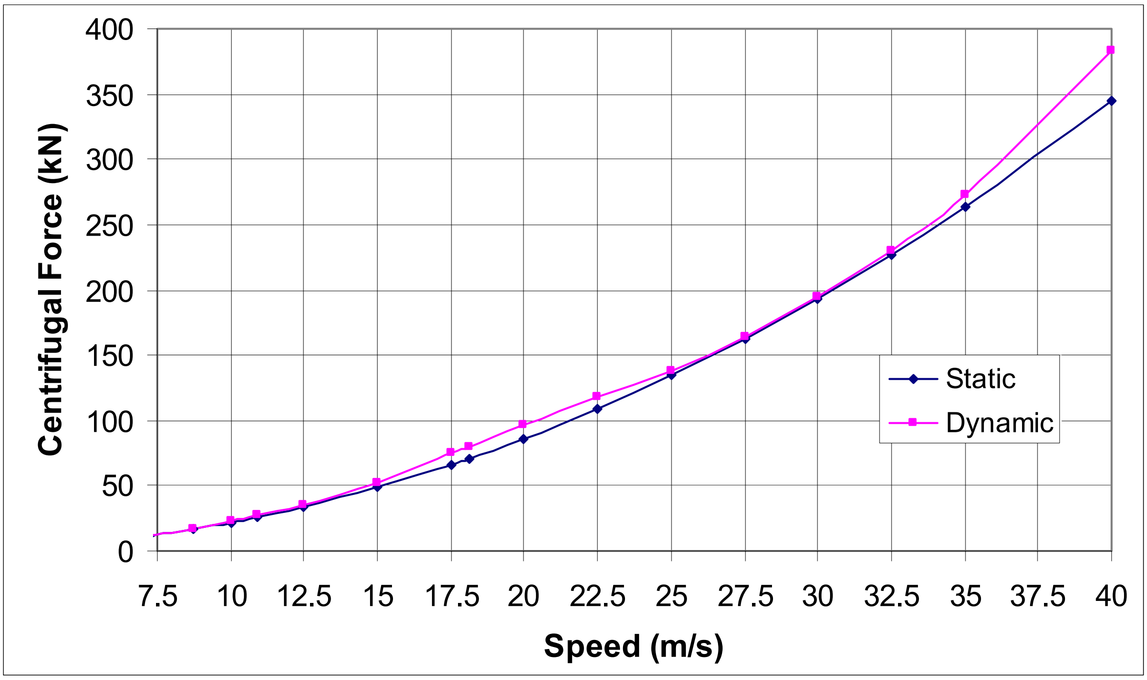
Group-1: Pier Dia = 1.5 m Pier Height = 8.0 m Acs = 2.1 m^2

Fig. 5.41 Centrifugal Force on Pier at Different train speeds on 24 m span



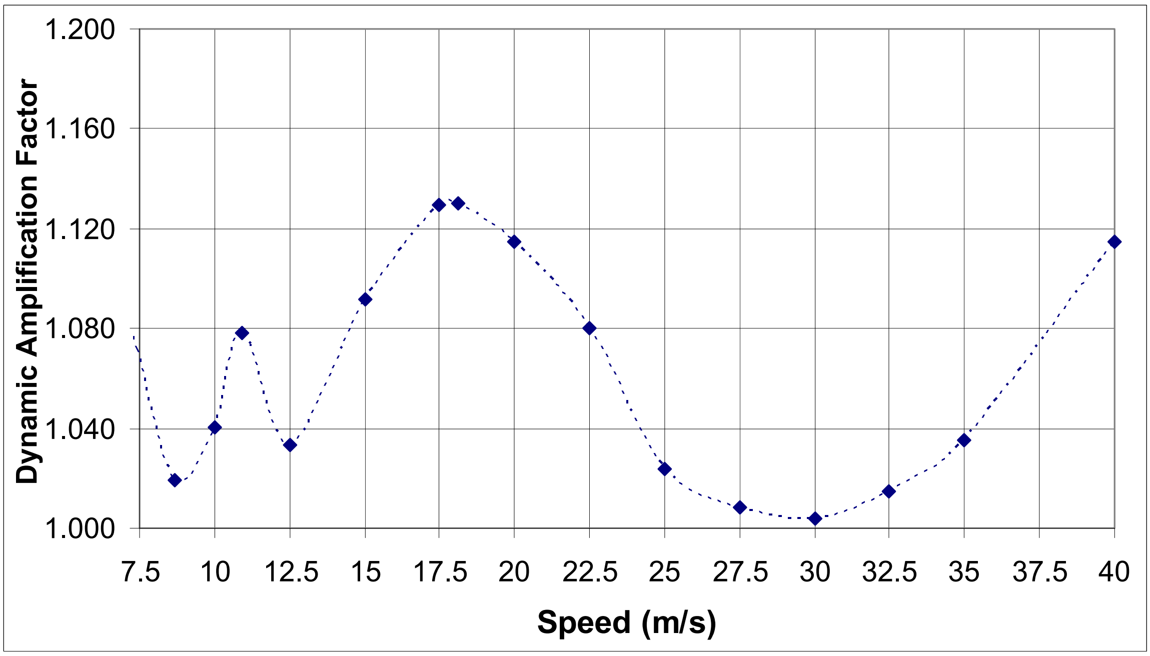
Group-1: Pier Dia = 1.5 m Pier Height = 8.0 m Acs = 2.1 m^2

Fig. 5.42 Dynamic Amplification Factor at Different train speeds for 24 m span



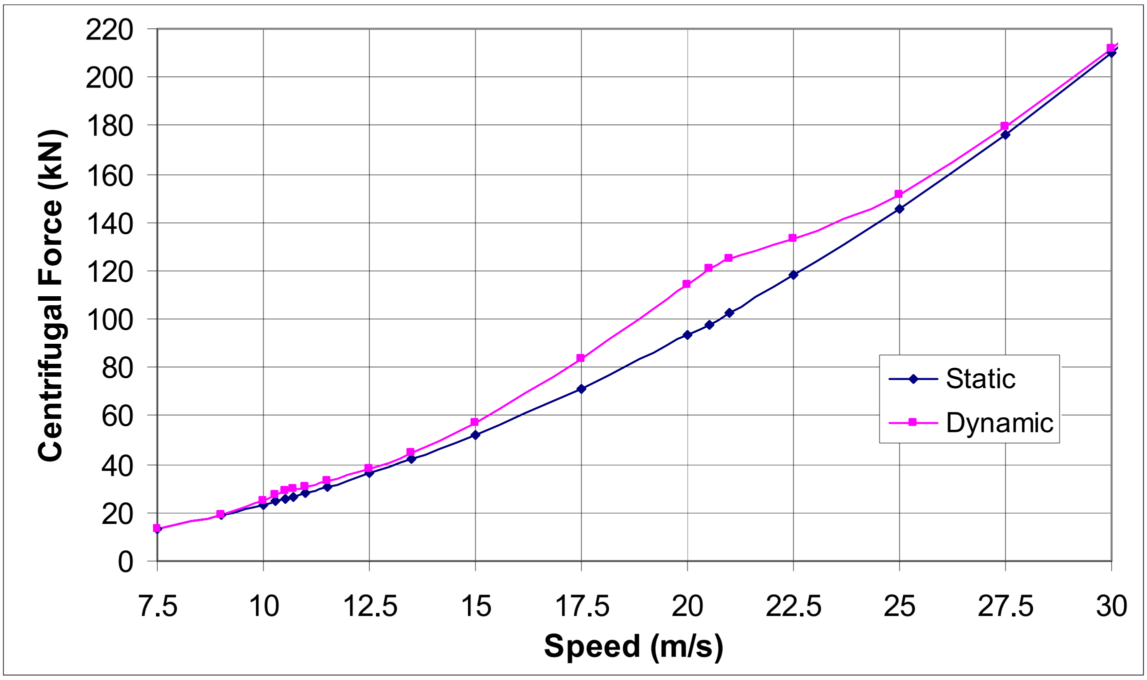
Group-1: Pier Dia = 1.5 m Pier Height = 8.0 m Acs = 2.1 m^2

Fig. 5.43 Centrifugal Force on Pier at Different train speeds on 26 m span



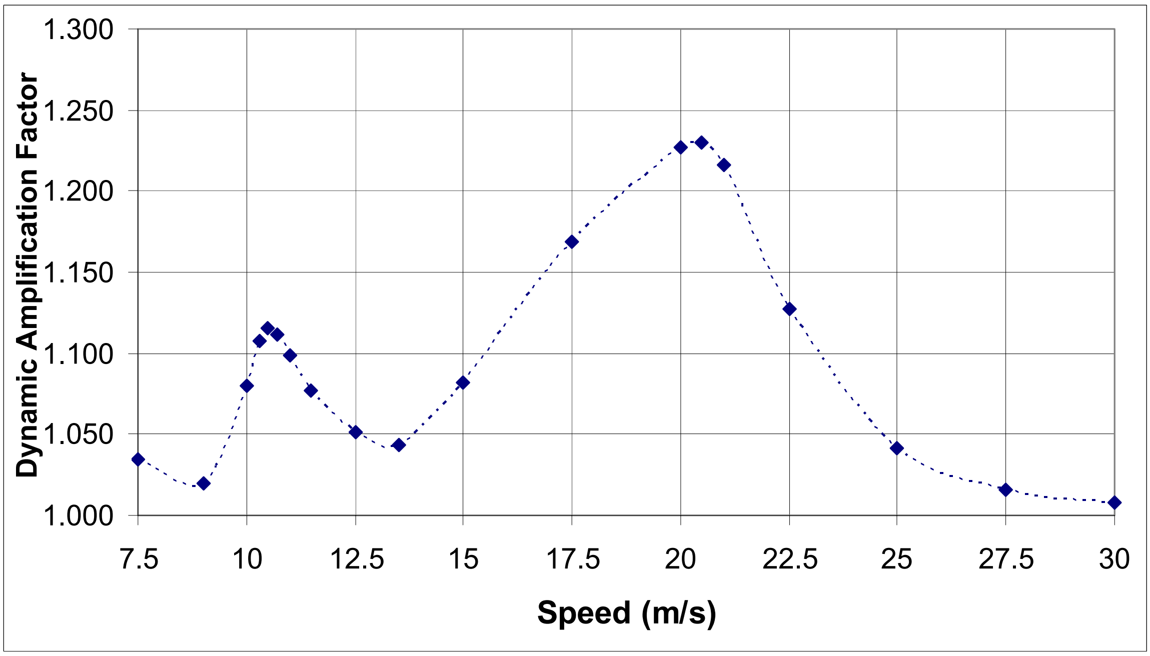
Group-1: Pier Dia = 1.5 m Pier Height = 8.0 m Acs = 2.1 m^2

Fig. 5.44 Dynamic Amplification Factor at Different train speeds for 26 m span



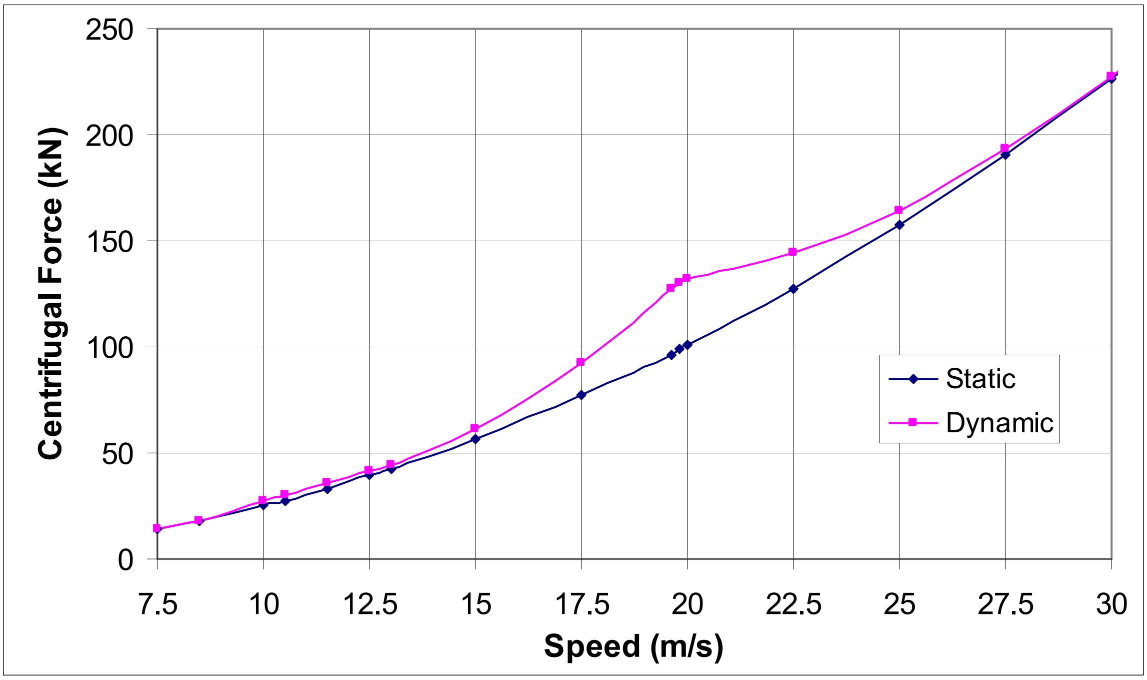
Group-1: Pier Dia = 1.5 m Pier Height = 8.0 m Acs = 2.1 m^2

Fig. 5.45 Centrifugal Force on Pier at Different train speeds on 28 m span



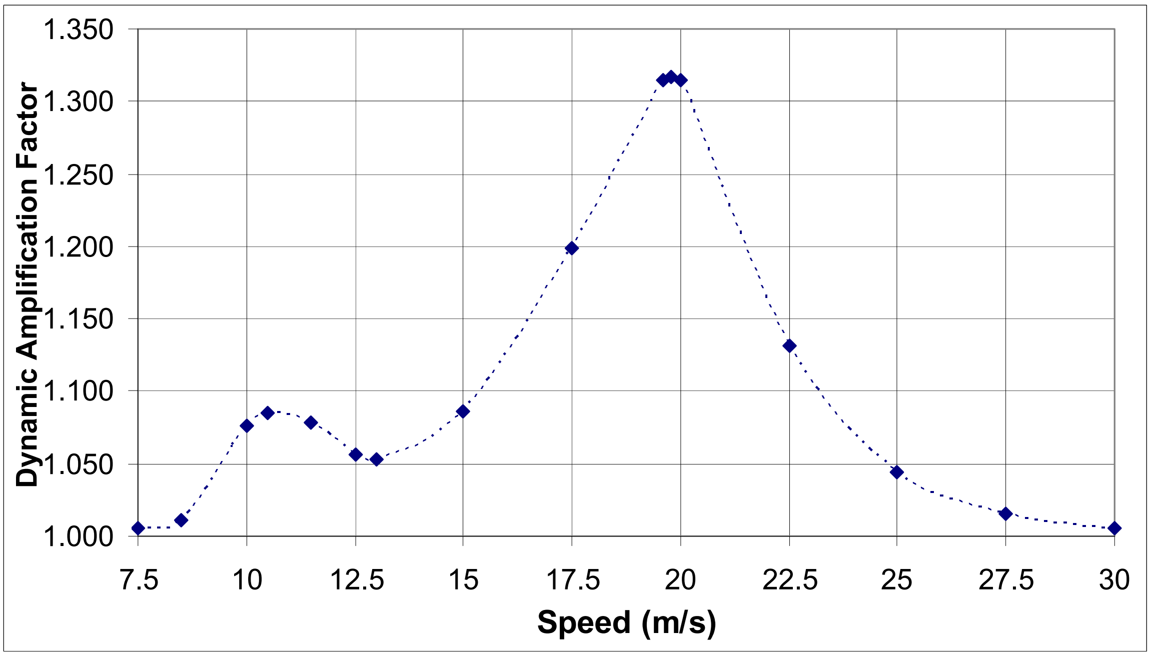
Group-1: Pier Dia = 1.5 m Pier Height = 8.0 m Acs = 2.1 m^2

Fig. 5.46 Dynamic Amplification Factor at Different train speeds for 28 m span



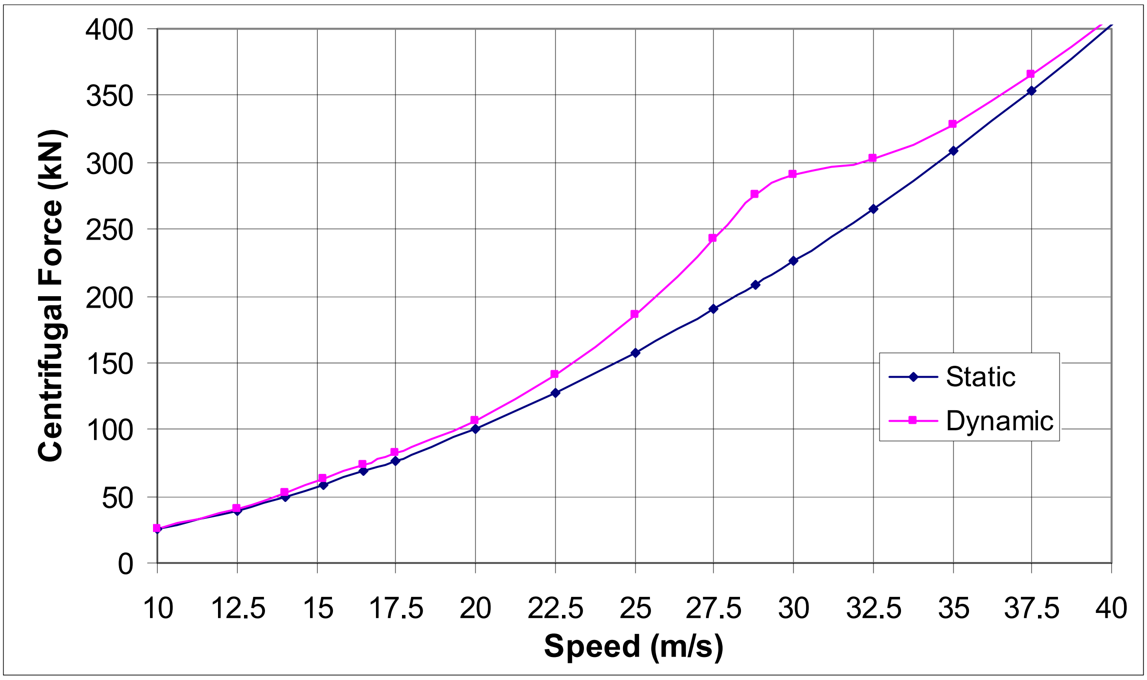
Group-1: Pier Dia = 1.5 m Pier Height = 8.0 m Acs = 2.1 m^2

Fig. 5.47 Centrifugal Force on Pier at Different train speeds on 30 m span



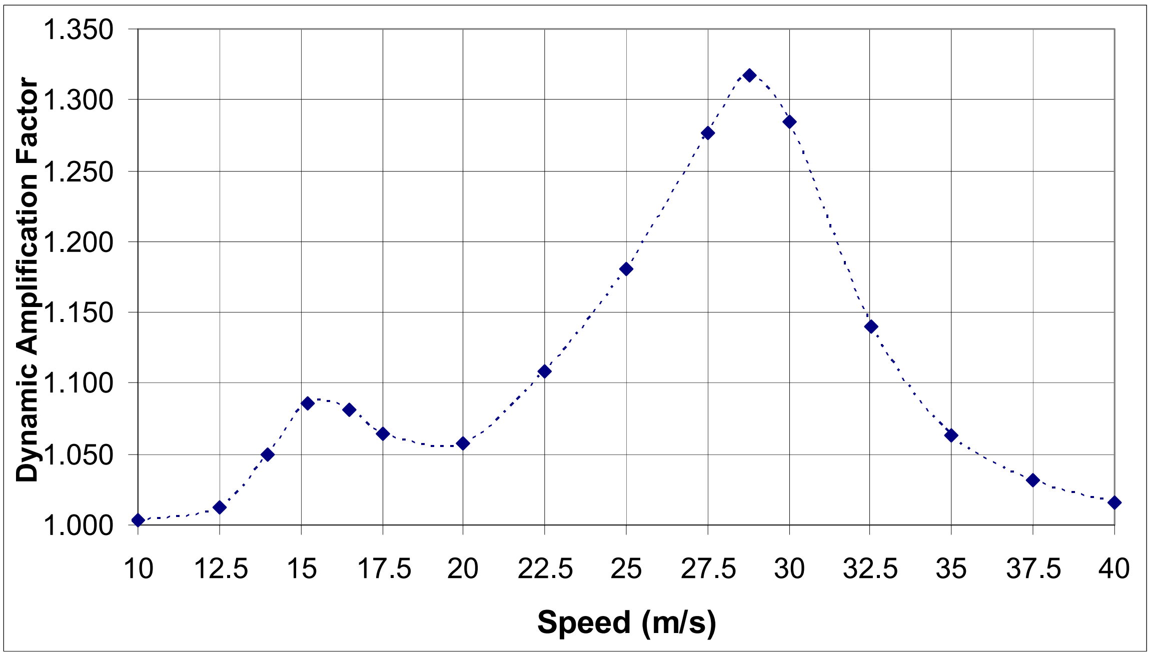
Group-1: Pier Dia = 1.5 m Pier Height = 8.0 m Acs = 2.1 m^2

Fig. 5.48 Dynamic Amplification Factor at Different train speeds for 30 m span



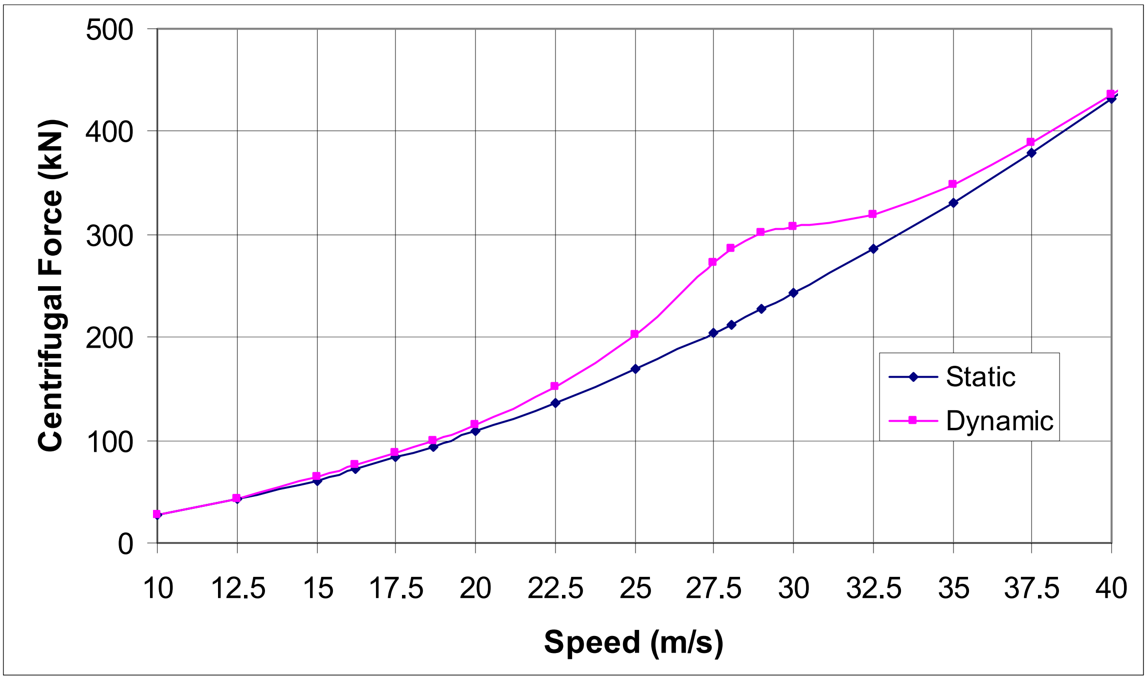
Group-4: Pier Dia = 2.2 m Pier Height = 10.0 m Acs = 3.3 m^2

Fig. 5.49 Centrifugal Force on Pier at Different train speeds on 30 m span



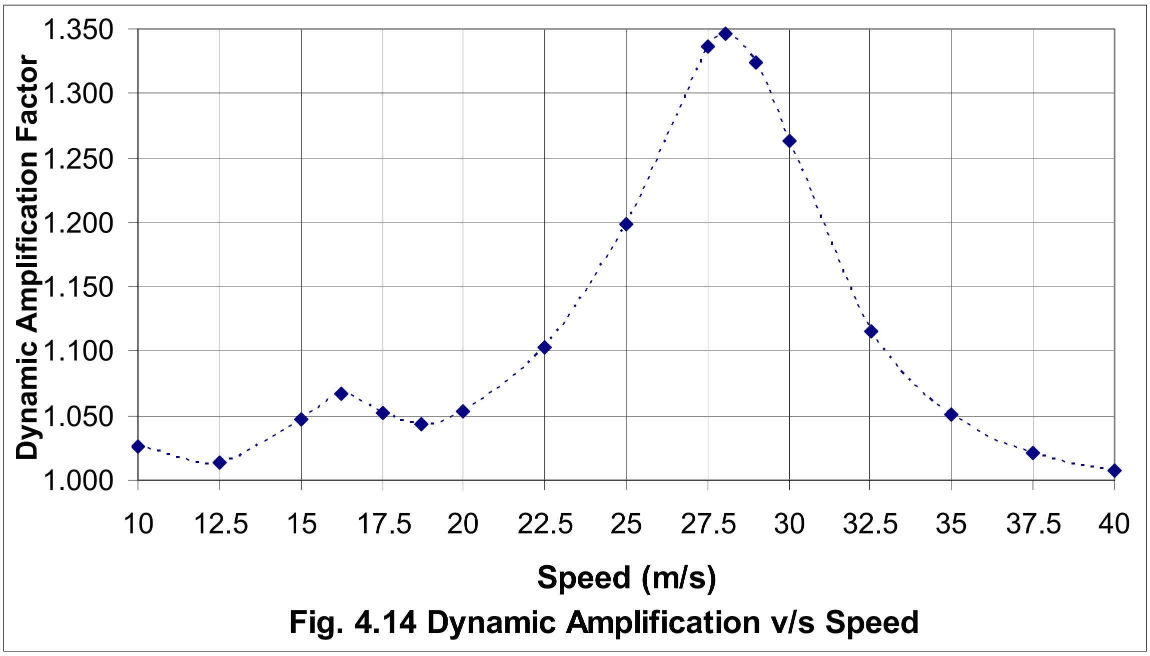
Group-4: Pier Dia = 2.2 m Pier Height = 10.0 m Acs = 3.3 m^2

Fig. 5.50 Dynamic Amplification Factor at Different train speeds for 30 m span



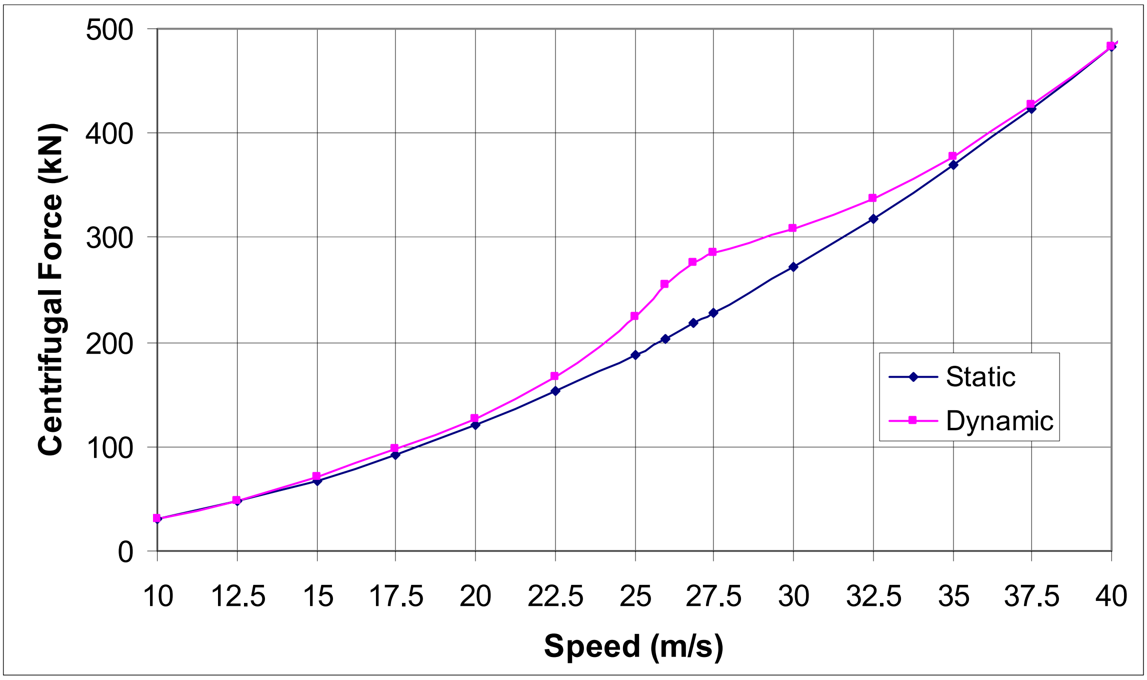
Group-4: Pier Dia = 2.2 m Pier Height = 10.0 m Acs = 3.3 m^2

Fig. 5.51 Centrifugal Force on Pier at Different train speeds on 32 m span



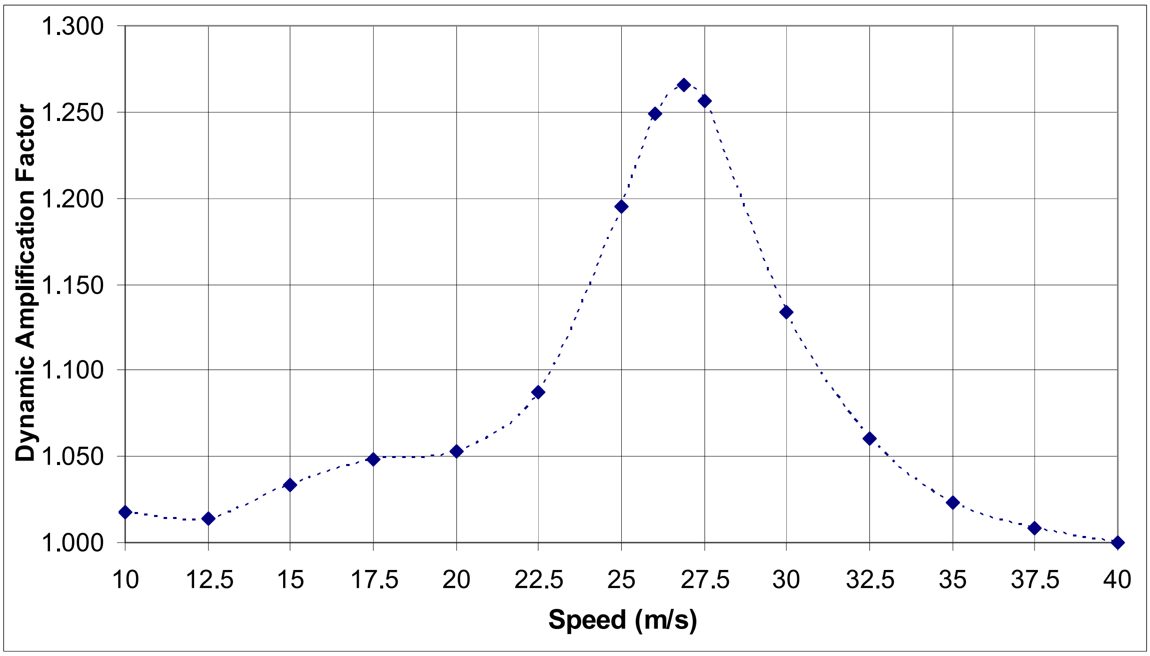
Group-4: Pier Dia = 2.2 m Pier Height = 10.0 m Acs = 3.3 m^2

Fig. 5.52 Dynamic Amplification Factor at Different train speeds for 32 m span



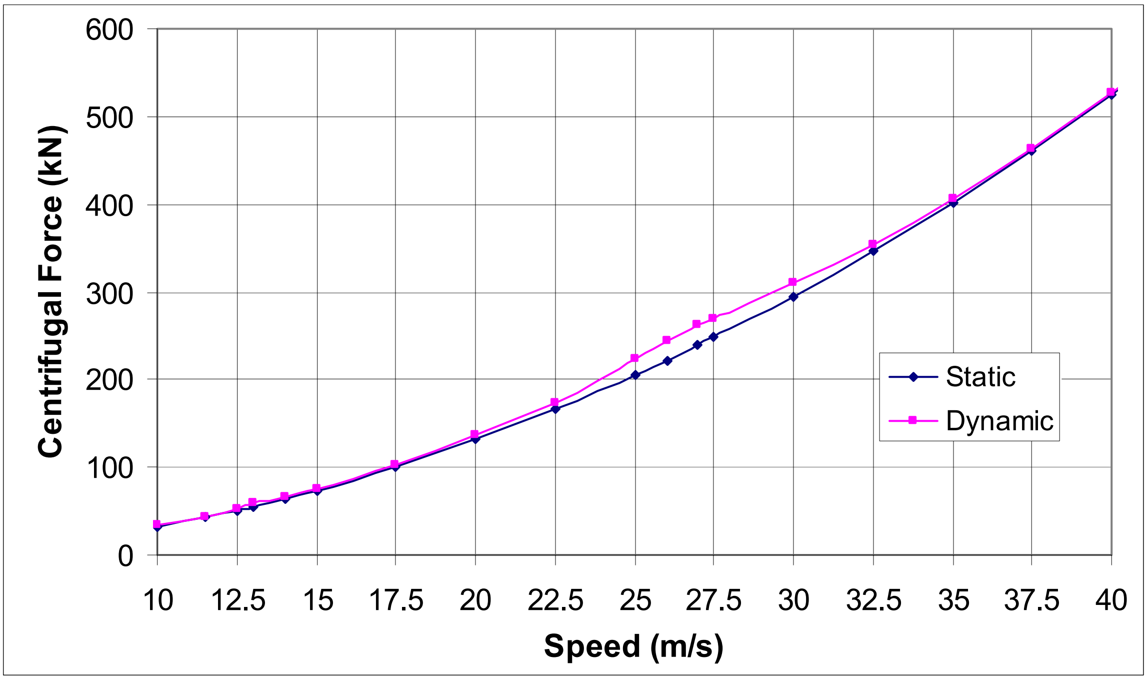
Group-4: Pier Dia = 2.2 m Pier Height = 10.0 m Acs = 3.3 m^2

Fig. 5.53 Centrifugal Force on Pier at Different train speeds on 36 m span



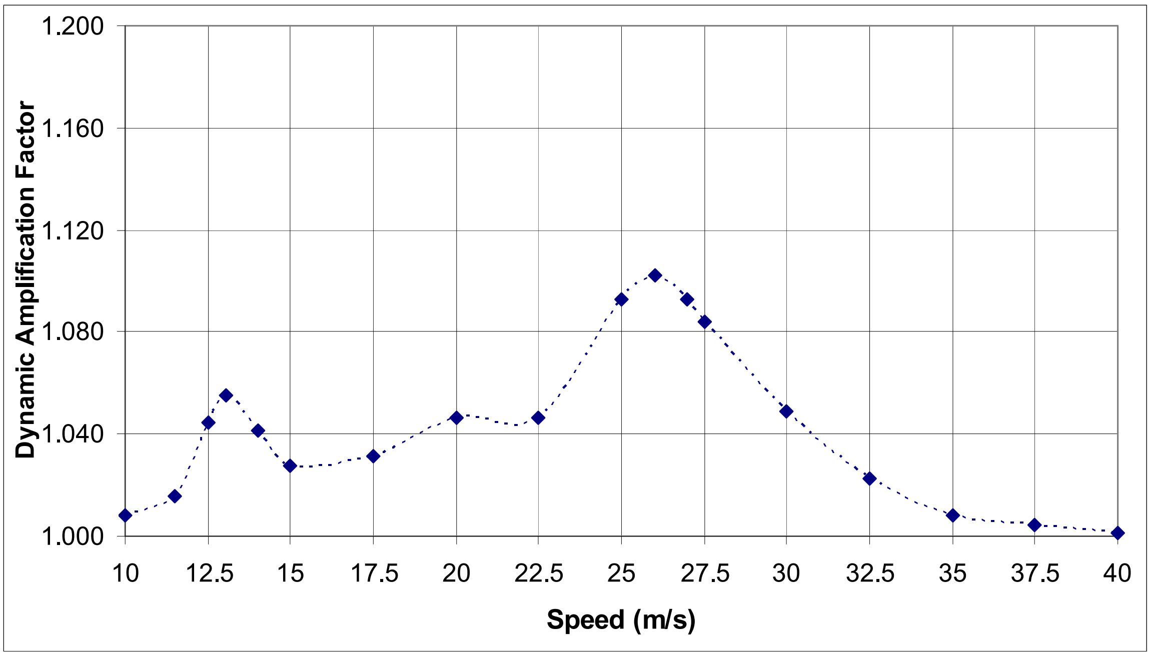
Group-4: Pier Dia = 2.2 m Pier Height = 10.0 m Acs = 3.3 m^2

Fig. 5.54 Dynamic Amplification Factor at Different train speeds for 36 m span



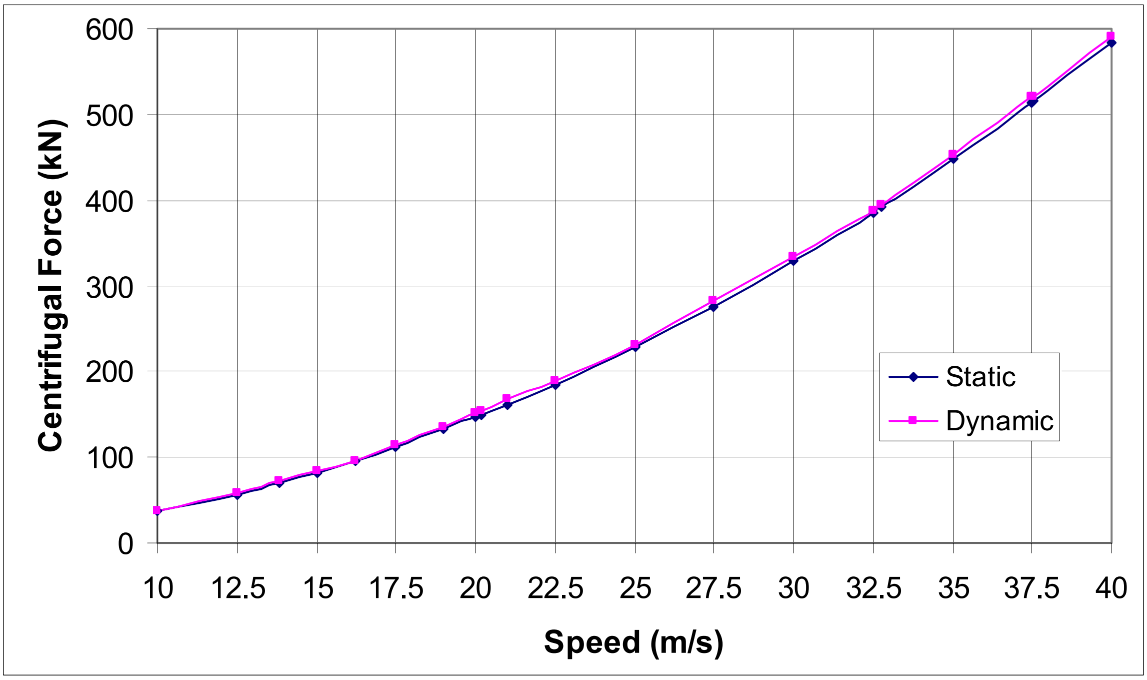
Group-4: Pier Dia = 2.2 m Pier Height = 10.0 m Acs = 3.3 m^2

Fig. 5.55 Centrifugal Force on Pier at Different train speeds on 40 m span



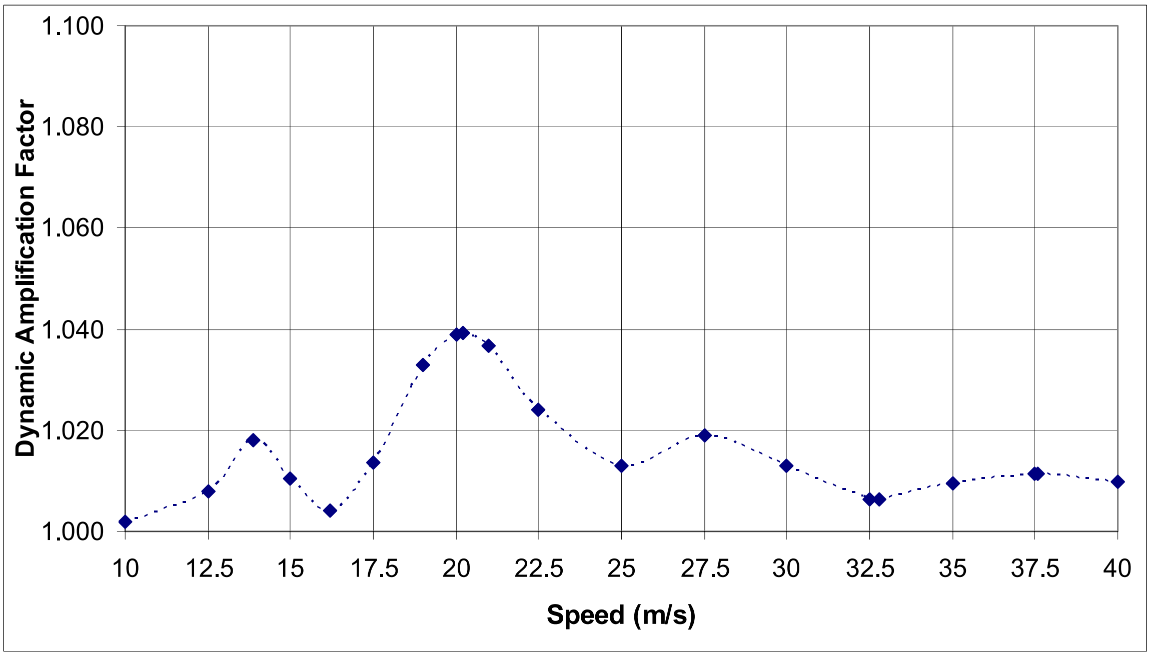
Group-4: Pier Dia = 2.2 m Pier Height = 10.0 m Acs = 3.3 m^2

Fig. 5.56 Dynamic Amplification Factor at Different train speeds for 40 m span



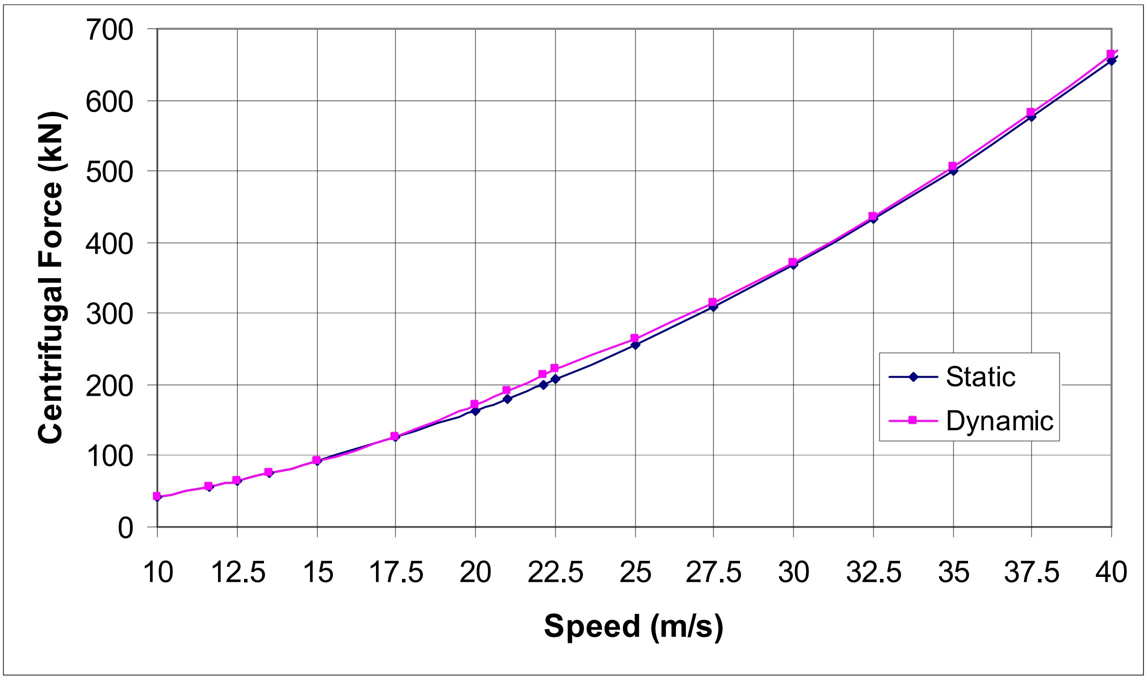
Group-4: Pier Dia = 2.2 m Pier Height = 10.0 m Acs = 3.3 m^2

Fig. 5.57 Centrifugal Force on Pier at Different train speeds on 45 m span



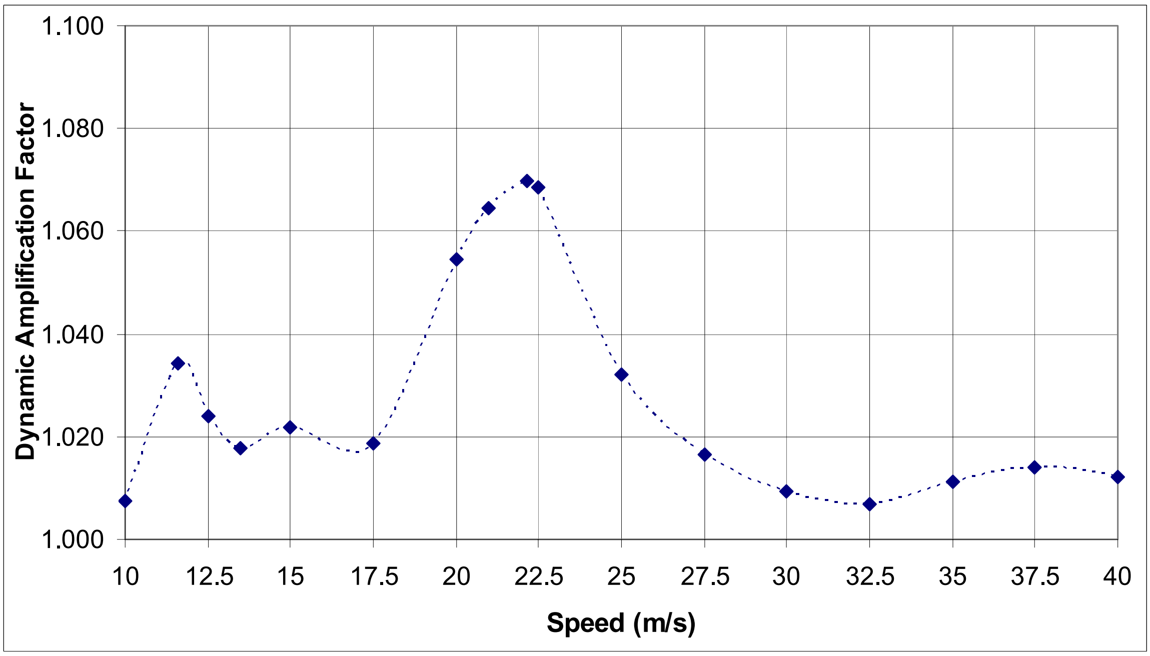
Group-4: Pier Dia = 2.2 m Pier Height = 10.0 m Acs = 3.3 m^2

Fig. 5.58 Dynamic Amplification Factor at Different train speeds for 45 m span



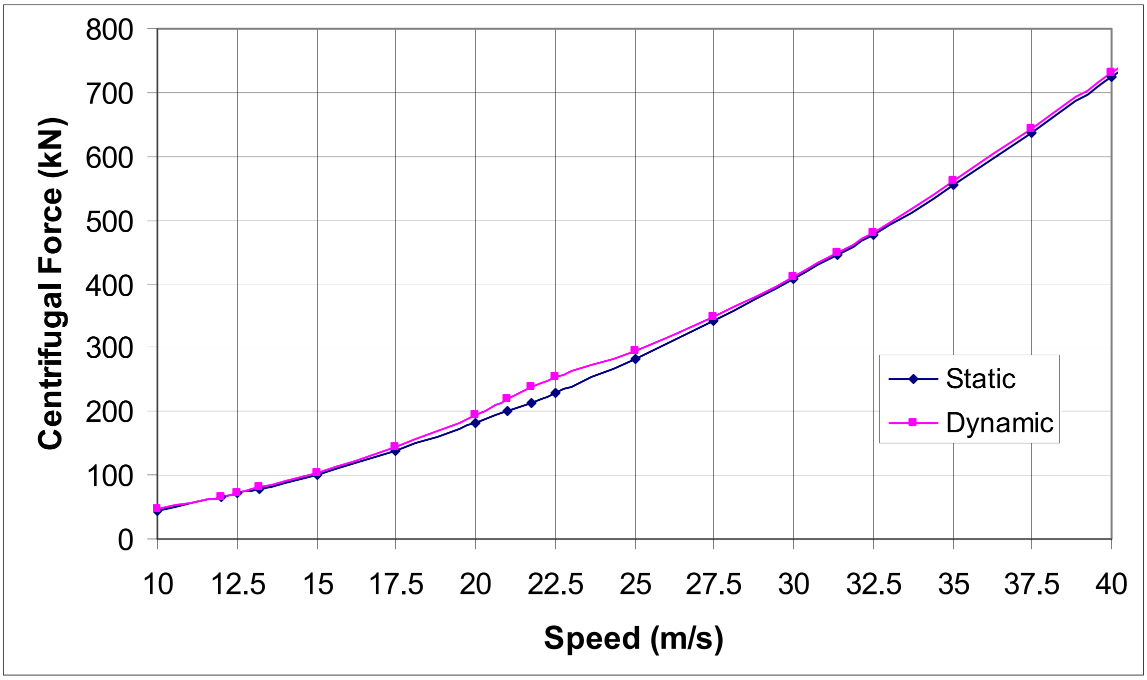
Group-4: Pier Dia = 2.2 m Pier Height = 10.0 m Acs = 3.3 m^2

Fig. 5.59 Centrifugal Force on Pier at Different train speeds on 50 m span



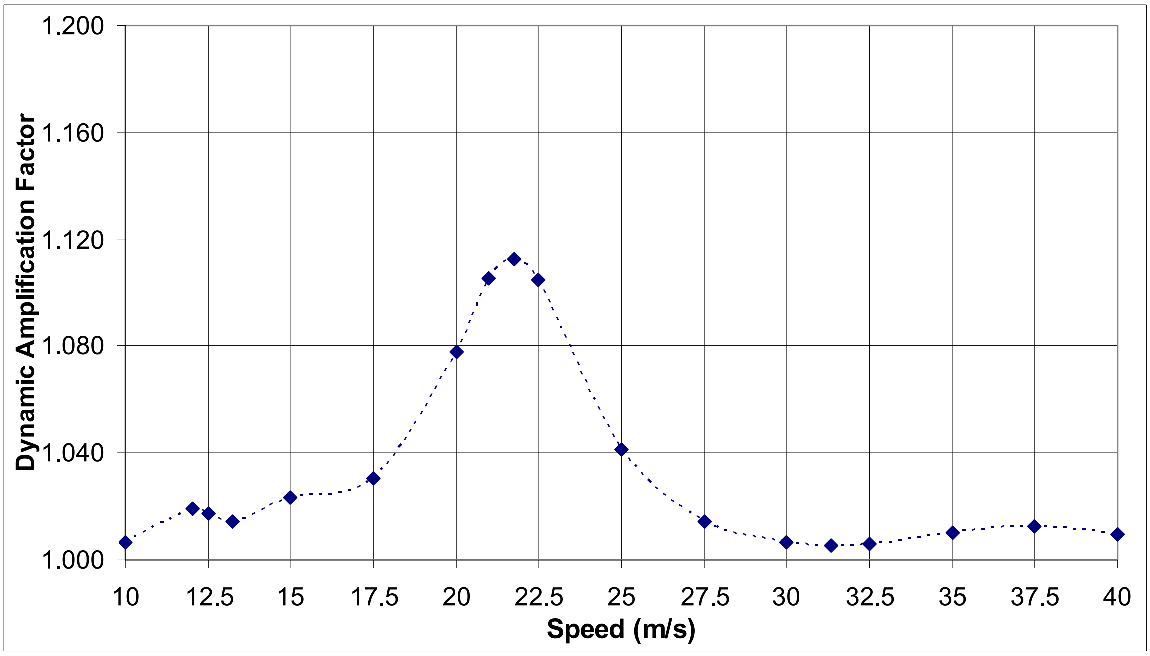
Group-4: Pier Dia = 2.2 m Pier Height = 10.0 m Acs = 3.3 m^2

Fig. 5.60 Dynamic Amplification Factor at Different train speeds for 50 m span



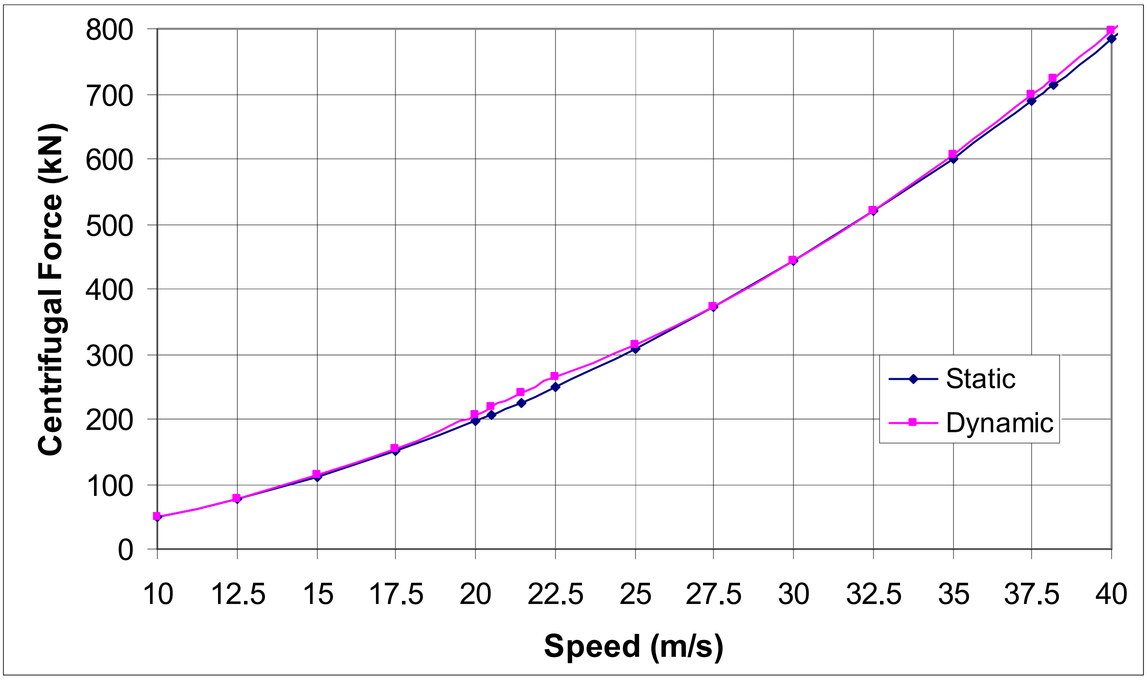
Group-4: Pier Dia = 2.2 m Pier Height = 10.0 m Acs = 3.3 m^2

Fig. 5.61 Centrifugal Force on Pier at Different train speeds on 55 m span



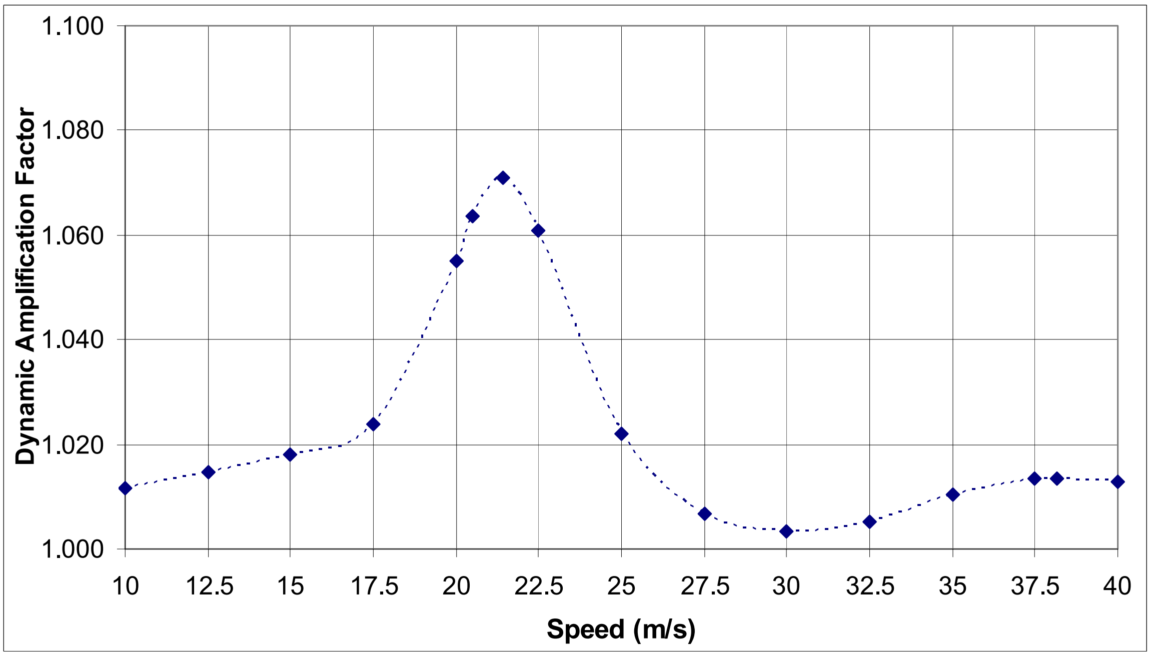
Group-4: Pier Dia = 2.2 m Pier Height = 10.0 m Acs = 3.3 m^2

Fig. 5.62 Dynamic Amplification Factor at Different train speeds for 55 m span



Group-4: Pier Dia = 2.2 m Pier Height = 10.0 m Acs = 3.3 m^2

Fig. 5.63 Centrifugal Force on Pier at Different train speeds on 60 m span



Group-4: Pier Dia = 2.2 m Pier Height = 10.0 m Acs = 3.3 m^2

Fig. 5.64 Dynamic Amplification Factor at Different train speeds for 60 m span

These figures present the dynamic behaviour of various spans over wide rage of train speed. From these figures too, it is observed that for shorter spans, maximum DAF is very high (> 2.5) which reduces as the span length approaches the length of one or multiple of coach length. Though the graph between Dynamic & Static centrifugal force and speed for longer spans does not differentiate much between them, the graph between DAF and speed clearly shows the difference. Multiple peaks in DAF curve can be noted in all the spans. The location (speed) of these peaks indicates the speed at which resonance occurs under passage of trains. The DAF and corresponding speed for the spans in Group-1 have been summarised in Table 5.9 given below. The variation of DAF with span has been presented in Fig. 5.65 for visual perception. Variation of DAF with span for Group-4 has been presented in Fig 5.66.

Table 5.9 Maximum DAF for Spans in Group-1

S.N.	Span (m)	Max DAF	Speed	DAF @ 20 m/s	DAF for <25 m/s
1	4	3.155	22.41	1.956	3.155
2	6	3.68	36.72	2.594	2.594
3	8	3.489	33.43	1.81	2.270
4	10	3.016	30.97	1.555	2.001
5	12	2.558	29.07	1.401	1.971
6	14	2.293	27.78	1.276	1.896
7	16	1.787	26.53	1.214	1.713
8	18	1.371	25.68	1.122	1.363
9	20	1.153	14.8	1.052	1.153
10	22	1.122	15.5	1.038	1.122
11	24	1.12	16.7	1.076	1.120
12	26	1.13	18.1	1.115	1.130
13	28	1.23	20.5	1.227	1.230
14	30	1.317	19.8	1.315	1.317

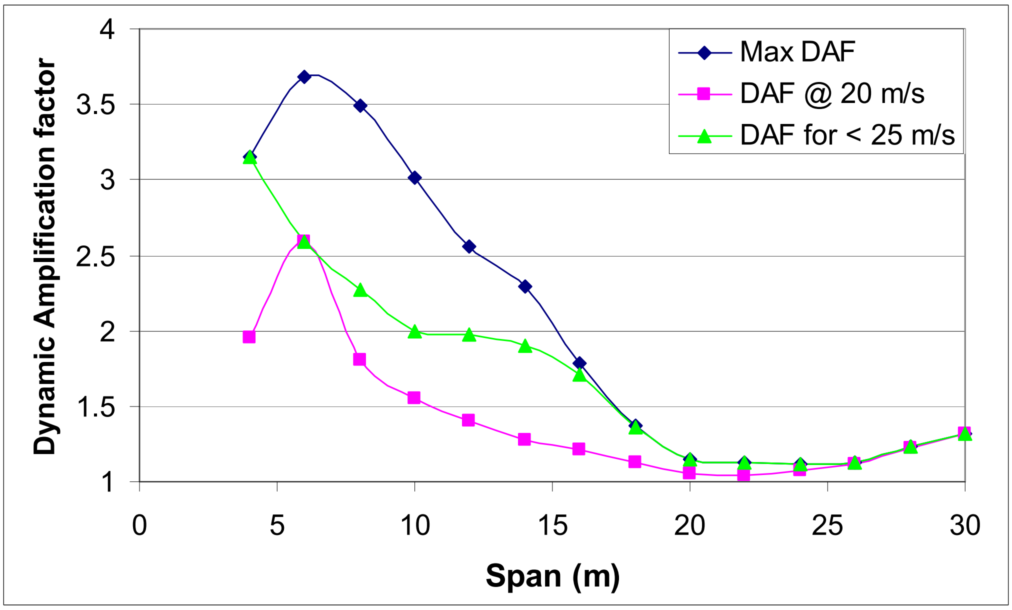


Fig. 5.65 Variation of DAF with span (Group-1)

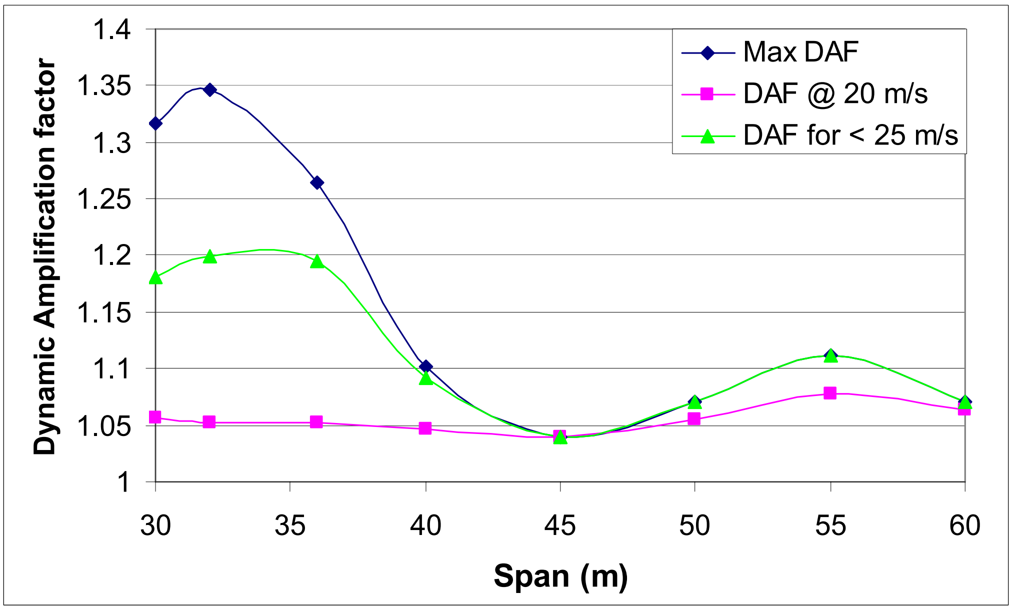


Fig. 5.66 Variation of DAF with span (Group-4)

From Fig. 5.65, it is observed that for shorter spans (< half coach length, i.e., 11 m), maximum DAF is very high (> 3.0) which reduces rapidly to almost unity as the span length approaches the coach length. Maximum DAF (3.68) occurs at about 7 m which is inter-coach bogie distance. Another peak appears at about 15 m which is intra-coach bogie distance. In Fig 5.66, two peaks have been observed for 32 and 55 m spans which correspond to odd multiple of half coach length, viz., 1.5, 2.5.

Further, one valley (DAF ≈ unity) has been obtained near 22 m span in Fig. 5.65 which correspond to the one coach length. Another valley has been obtained near 45 m span in Fig. 5.66 which corresponds to twice the coach length. The trend of graph in Fig. 5.66 suggests the presence of another valley near 65 m span which will correspond to thrice the coach length. Hence, minimum DAF (≈ unity) occurs when span length is in multiple of one coach length, viz., 1, 2, 3 etc.

Table 5.10 presents the dynamic centrifugal force for spans in Group-1 at different speeds. Static centrifugal force for 22 m span has also been listed therein.

Table 5.10 Dynamic CF for Spans in Group-1 at various Speeds

S.N.	**Span (m)**	Dyn CF @ 10 m/s	Dyn CF @ 20 m/s	Dyn CF @ 30 m/s
1	4	9.58	52.98	160.85
2	6	9.46	85.82	196.76
3	8	10.46	71.41	261.47
4	10	13.47	73.33	315.53
5	12	17.1	72.46	293.69
6	14	14.9	70.04	253.54
7	16	14.56	70.07	189.68
8	18	15.9	69.14	162.18
9	20	17.06	69.98	157.62
10	22	18.06	74.64	164.95
11	24	19.75	84.61	177.48
12	26	22.37	95.90	194.29
13	28	25.18	144.40	211.41
14	30	27.07	132.28	227.68
15	Static CF 22	17.98	74.64	164.95

The variation of dynamic centrifugal force with span at different train speeds has been presented in Fig. 5.67. Variation of dynamic centrifugal force with span for Group-4 has been presented in Fig 5.68.

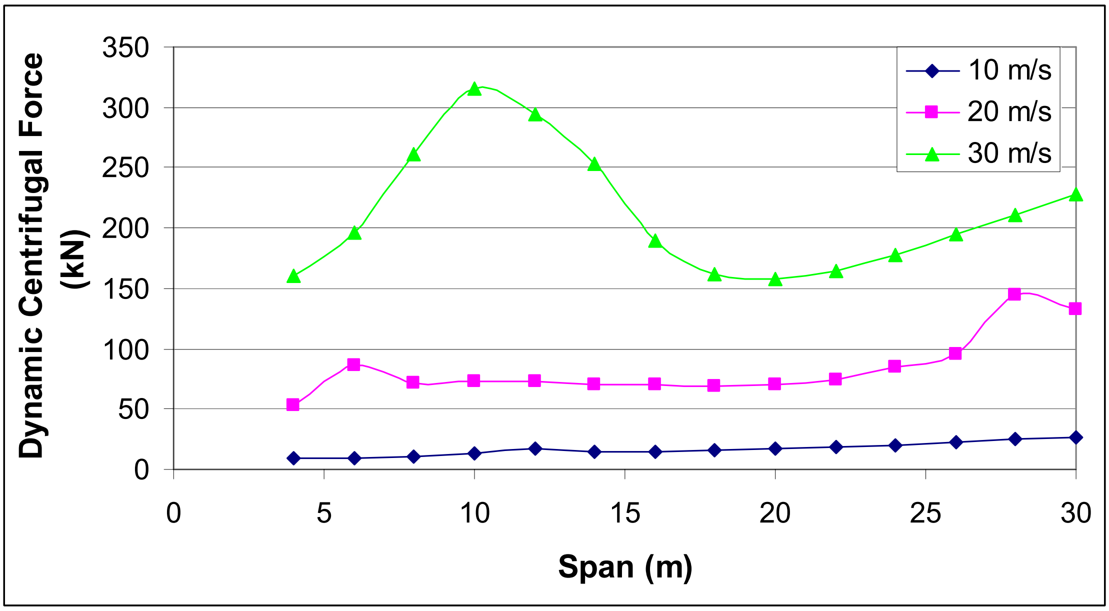


Fig. 5.67 Variation of Dynamic Centrifugal Force with span (Group-1)

From Fig. 5.67, it has noted that for shorter spans (< one coach length, 22 m) at lower speeds (10 & 20 m/s), the dynamic centrifugal force is almost constant irrespective of speed. However, at higher speed (30 m/s), the dynamic centrifugal force for shorter spans (10 m) is very high (≈ 2 times of static force on 22 m span, i.e., one coach length).

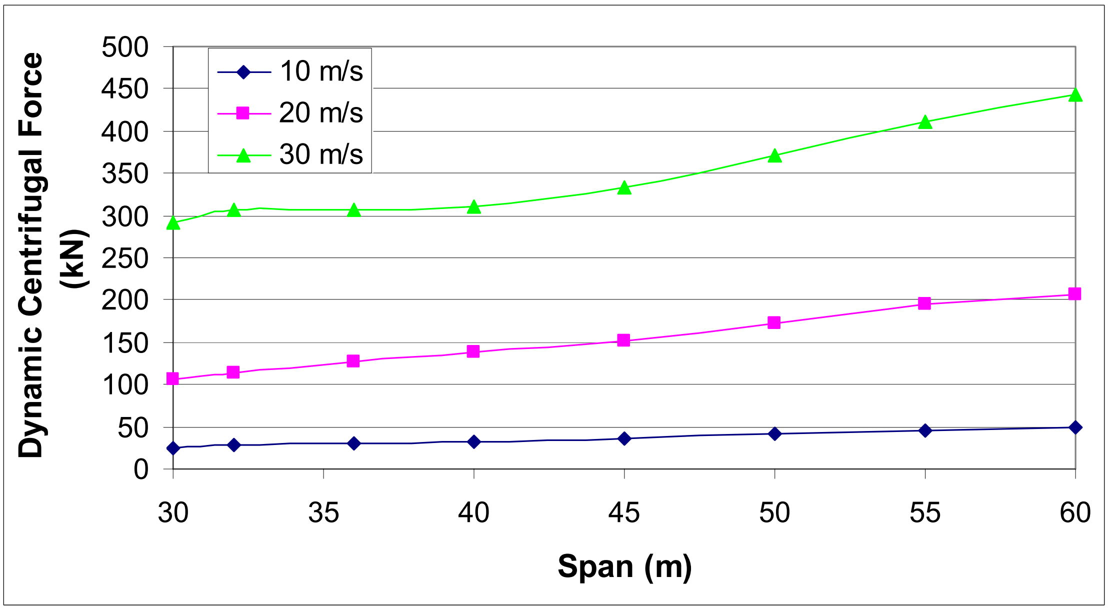


Fig. 5.68 Variation of Dynamic Centrifugal Force with span (Group-4)

Fig 5.68 suggests that for longer spans, centrifugal force increases gradually with span, the rate being higher at higher speeds. There is no prominent peak as was observed in Fig 5.67. This is in conformity of observations made earlier that Dynamic amplification is more prominent in shorter spans.

From the discussion presented above on response of pier for different spans, following conclusions can be drawn at this stage:

i) Significant dynamic amplification takes place in shorter spans (length less than half coach length).

ii) For spans less than one coach length (< 22 m), Dynamic centrifugal force is almost constant at lower speeds (upto 20 m/s) irrespective of speed and approximately equal to static centrifugal force for 22 m span.

iii) For spans less than one coach length (< 22m), Dynamic centrifugal force is very high at higher speeds (30 m/s). For 10 m span, it is approximately double in comparison to static centrifugal force for 22 m span.

iv) The dynamic amplification is much less in longer spans, maximum DAF being 1.35 for 32 m span.

v) Maximum DAF occurs when span length is equal to inter-coach bogie distance (≈ 7 m). Other peaks occurs when coach length is in odd multiple of half coach length, viz., 1.5, 2.5 etc. The value of maximum DAF decreases for such peaks with increase in span.

vi) Minimum DAF (≈ unity) occurs when span length is in multiple of coach length, viz., 1, 2, 3 etc.

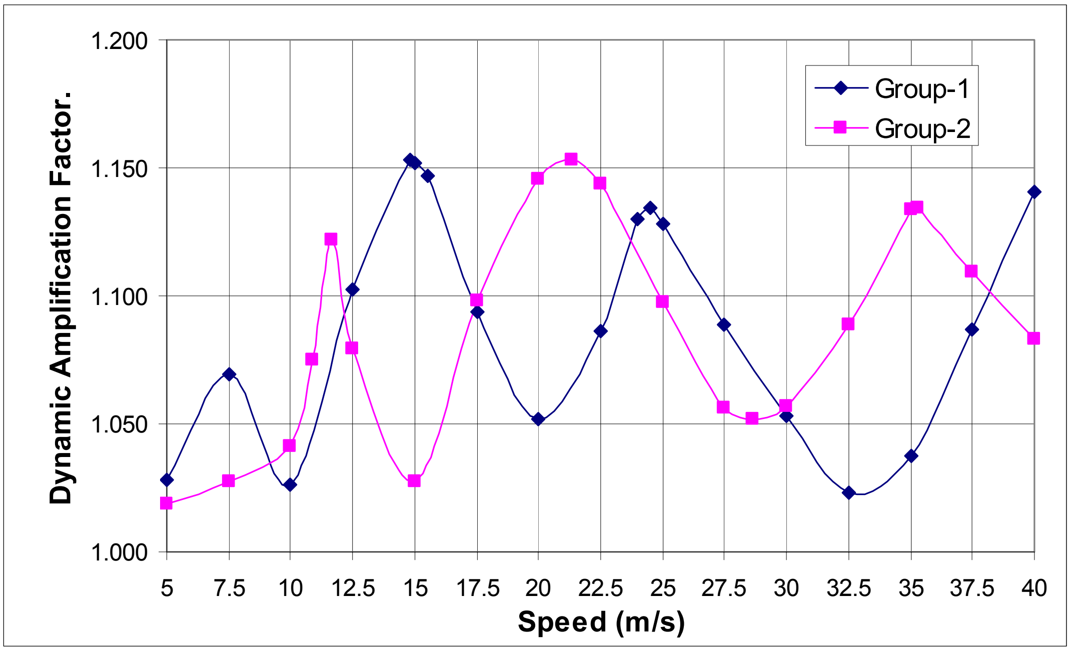
To further study the dynamic behaviour of pier for different system parameters, a parametric study is required which has been carried out and presented in next article.

5.5 PARAMETRIC STUDY ON DYNAMIC BEHAVIOUR OF BRIDGE PIERS

To study the effect of variation in various parameters considered in this work, viz., pier dimensions (dia and height) and weight of span per meter; all the spans given in Table 5.1 for Group 2, 3 & 5 have also been analysed. The spans given in Table 5.1 under various groups have further been rearranged so as to group them in such a way that effect of variation in one parameter at a time can be studied. This will be discussed in subsequent text.

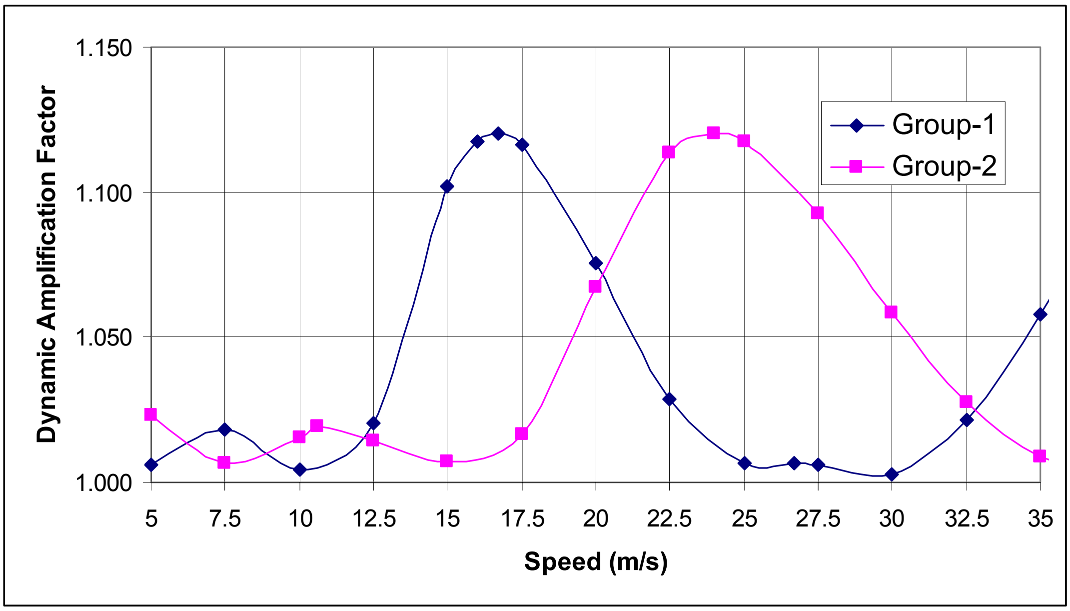
5.5.1 Effect of variation of Pier Dia on DAF for a Span

The effect of change in dia of pier is to change the stiffness of pier. Among the various groups analysed, configuration of Group-1 and Group-2 are same except for change in pier Dia. In these two groups, spans from 20 to 30 m are common. Hence, for studying the effect of change in pier dia on pier response, results for 20, 24, 26 and 30 m spans of groups 1 & 2 have been compared. Fig. 5.69 to 5.72 presents the superimposed dynamic amplification factor for these spans for both the groups.



Pier Dia Group-1 : 1.5 m Group-2 : 1.8 m

Fig. 5.69 Variation of Dynamic Amplification Factor for 20 m Span

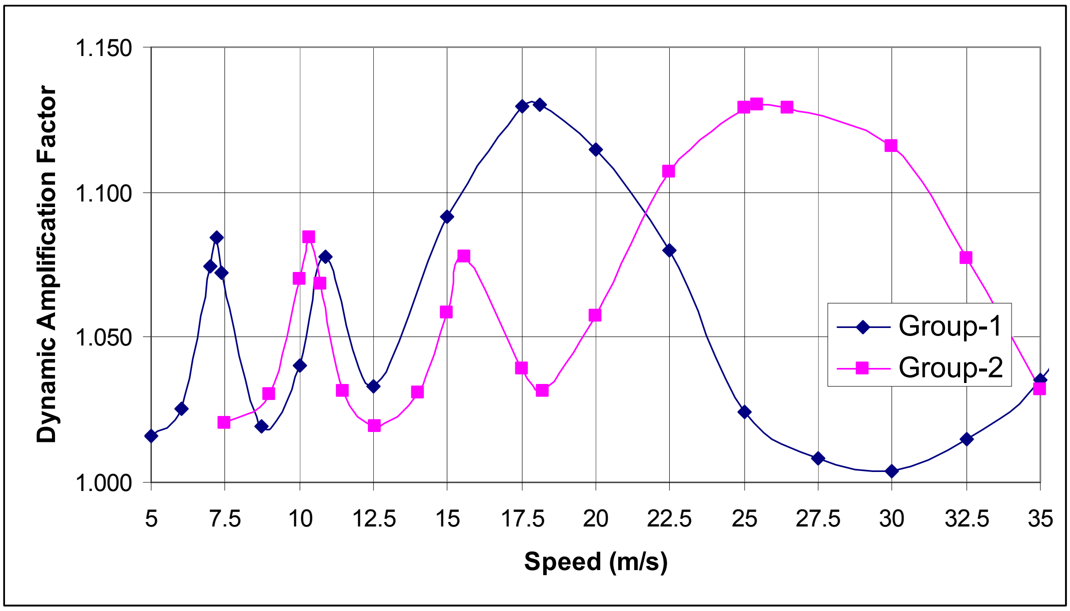


Pier Dia

Group-1 : 1.5 m

Group-2 : 1.8 m

Fig. 5.70 Variation of Dynamic Amplification Factor for 24 m Span

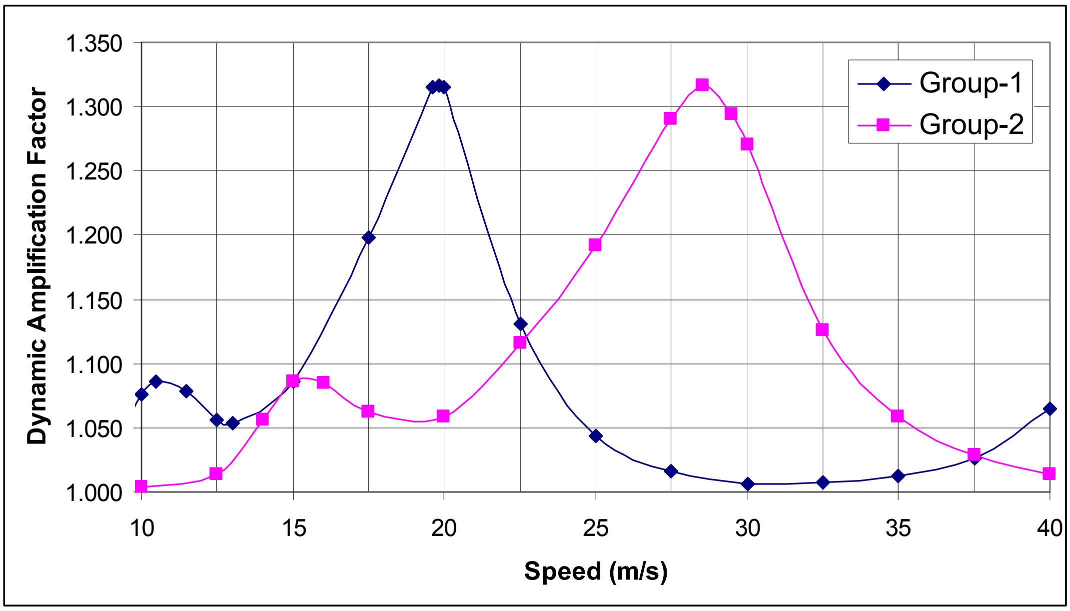


Pier Dia

Group-1 : 1.5 m

Group-2 : 1.8 m

Fig. 5.71 Variation of Dynamic Amplification Factor for 26 m Span



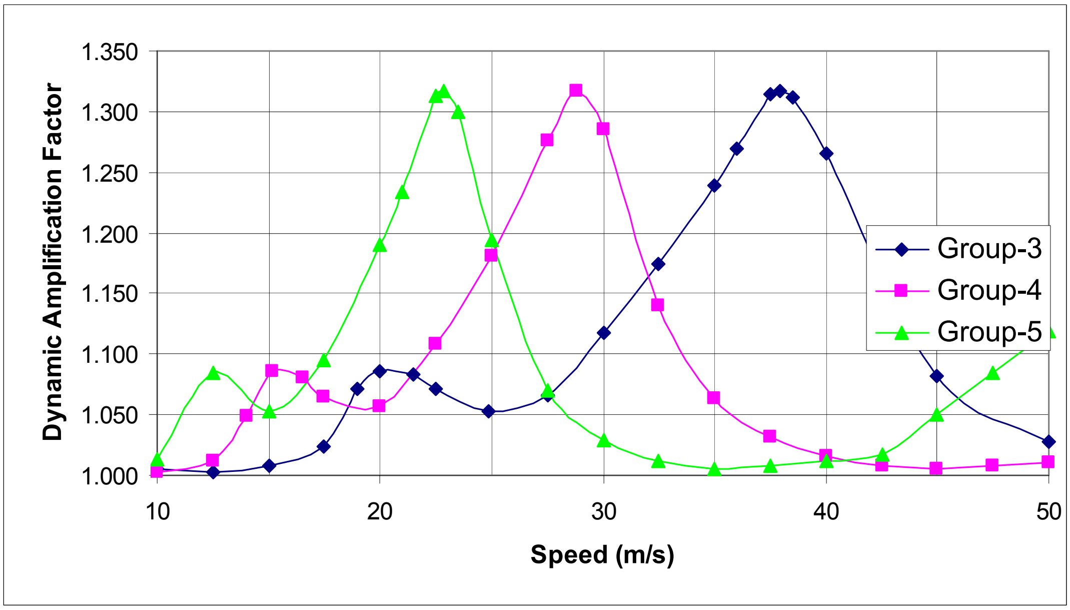
Pier Dia Group-1 : 1.5 m Group-2 : 1.8 m

Fig. 5.72 Variation of Dynamic Amplification Factor for 30 m Span

From the figures, it has been noted that due to change in dia of pier, the speed at which maximum dynamic amplification occurs (i.e. resonance) changes, however, the value of maximum DAF remains unchanged. As already discussed, the effect of change in pier dia is to change the stiffness of system thus altering the natural time period of the structure. Due to change in natural time period, the speed at which resonance occur changes and hence shift in DAF peak is observed.

5.5.2 Effect of variation of Pier Height on DAF for a Span

The effect of change in height of pier is also to change the stiffness of pier. Among various groups analysed, configuration of Group-3, Group-4 and Group-5 are same except for change in pier height. In these two groups, spans from 30 to 60 m are common. Hence, for studying the effect of change in pier height on pier response, results for 30, 40 and 55 m spans of groups 3, 4 & 5 have been compared. Fig. 5.73 to 5.75 presents the superimposed dynamic amplification factor for these spans for group 3, 4 & 5.



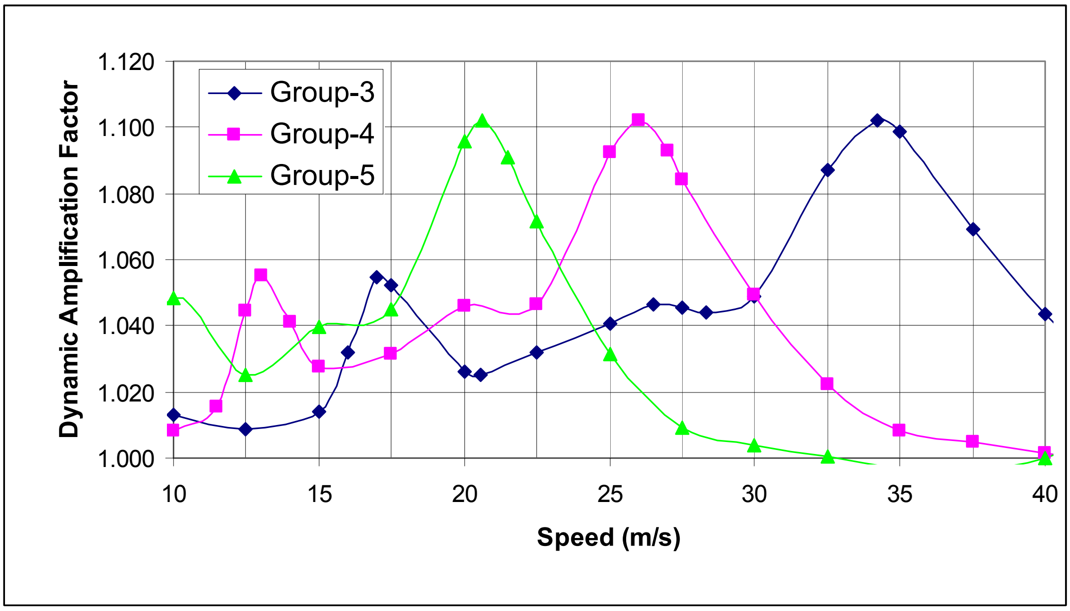
Pier Height

Group-3 : **8 m**

Group-4 : **10 m**

Group-5 : **12 m**

Fig. 5.73 Variation of Dynamic Amplification Factor for 30 m Span



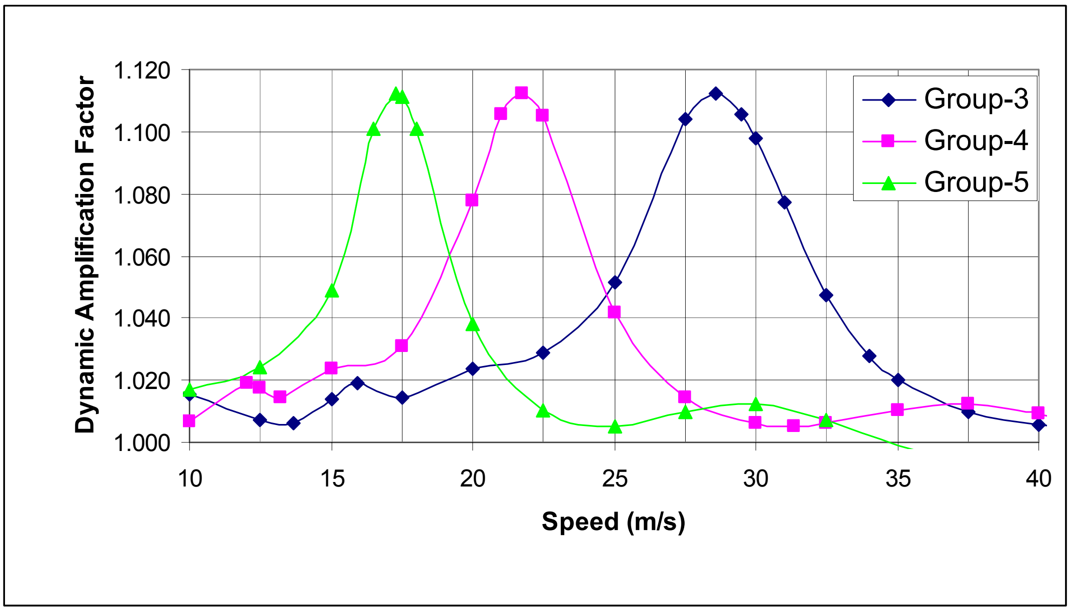
Pier Height

Group-3 : **8 m**

Group-4 : **10 m**

Group-5 : **12 m**

Fig. 5.74 Variation of Dynamic Amplification Factor for 40 m Span



Pier Height Group-3 : 8 m Group-4 : 10 m Group-5 : 12 m

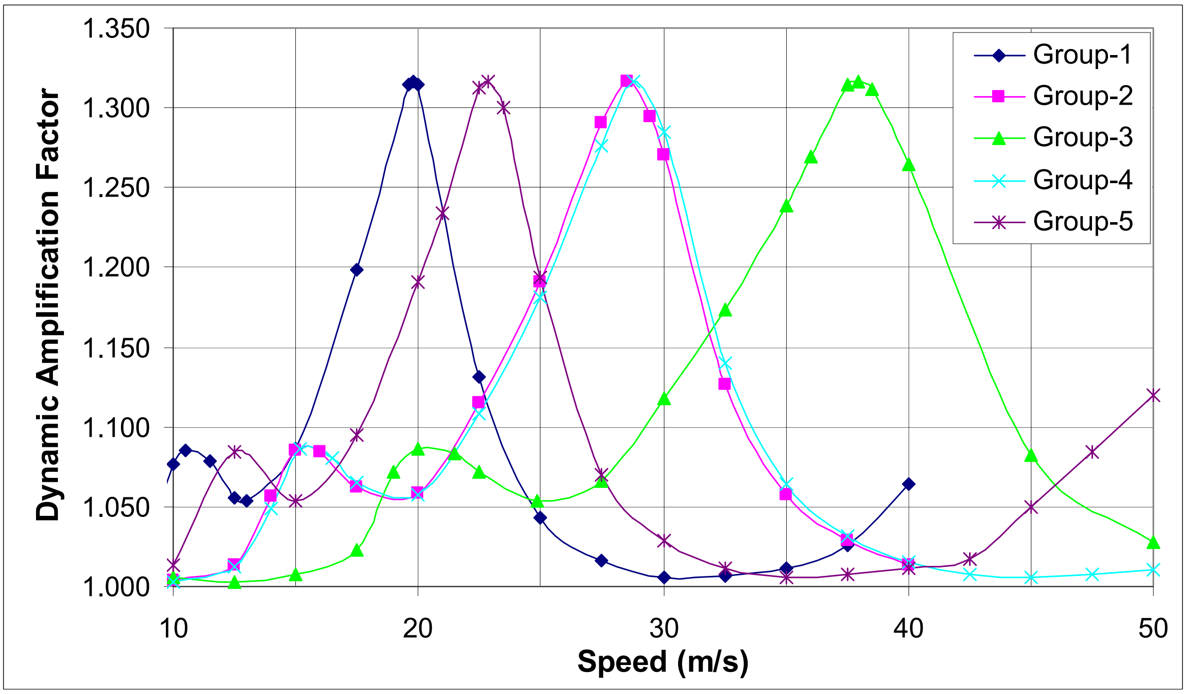
Fig. 5.75 Variation of Dynamic Amplification Factor for 55 m Span

From the figures, it has been noted that due to change in height of pier, the speed at which maximum dynamic amplification occur (i.e. resonance) changes, however, the value of maximum DAF remains unchanged. Similar to change in dia of piers, the effect of change in pier height is also to change the stiffness of system thus altering the natural time period of the structure. Due to change in natural time period, the speed at which resonance occur changes and hence shifts in DAF peak is observed.

5.5.3 Effect of variation of Weight of Span/m on DAF for a Span

The effect of change in weight of span is to change the mass of the system. Among various groups analysed, cross-sectional area and weight/m of span are same for Groups-1 and 2. The weight per m is also same for Group-3, 4 and 5. In all these groups, 30 m span is common. The effect of change in pier dia and height has already been studied. Hence, for studying the effect of change in span weight on pier response, results for 30 m spans of groups 1 to 5 have been compared.

Fig. 5.76 presents the superimposed dynamic amplification factor for 30 m span for all the 5 groups.



	Pier Height	Pier Dia	Span Weight/m
Group-1 :	**8 m**	**1.5 m**	**5.25 MT/m**
Group-2 :	**8 m**	**1.8 m**	**5.25 MT/m**
Group-3 :	**8 m**	**2.2 m**	**8.25 MT/m**
Group-4 :	**10 m**	**2.2 m**	**8.25 MT/m**
Group-5 :	**12 m**	**2.2 m**	**8.25 MT/m**

Fig. 5.76 Variation of Dynamic Amplification Factor for 30 m Span

From the figure, it has been noted that due to change in all three parameters, viz., pier dia, height and weight of span; the speed at which maximum dynamic amplification occurs (i.e. resonance) changes, however, the value of maximum DAF remains unchanged. The effect of change in span weight is to change the mass of system thus altering the natural time period of the structure. Due to change in natural time period, the speed at which resonance occur changes and hence shift in DAF peak is observed. Hence, it prima-facie appears that maximum DAF is a function of span only. To confirm this, results for all groups have been superimposed.

Figure 5.77 presents the variation of maximum dynamic amplification factor (obtained for speed at which maxima of DAF occurs) with span for all 5 groups.

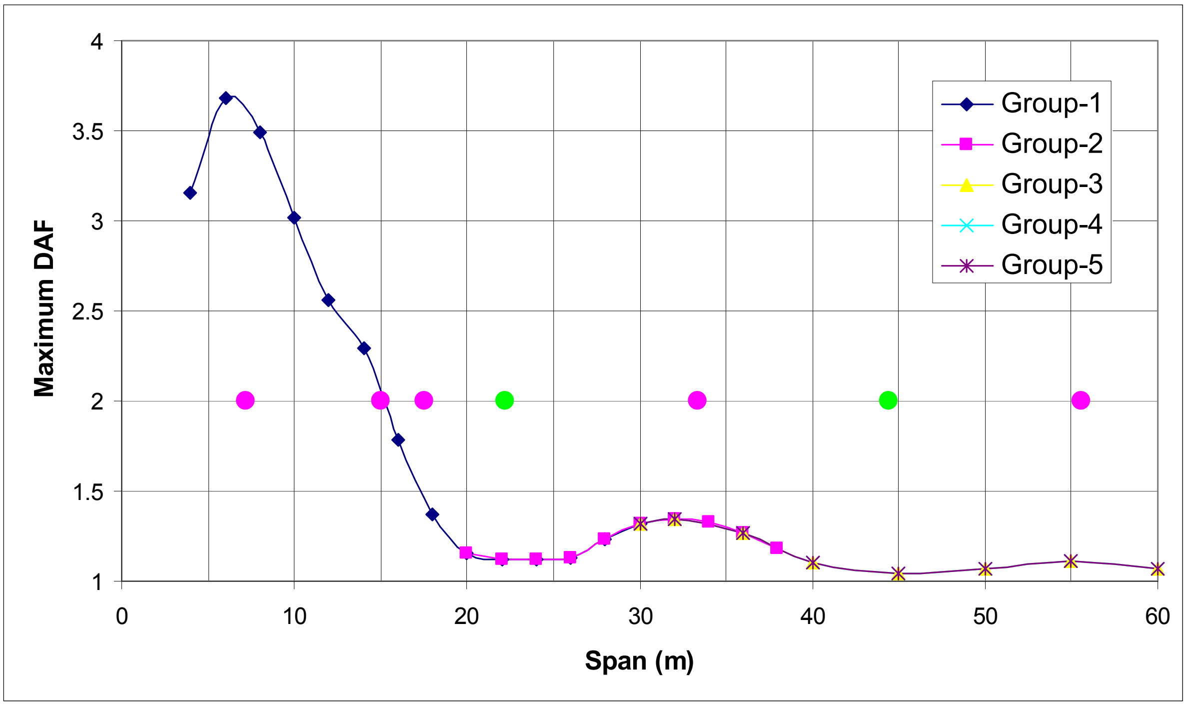


Fig. 5.77 Variation of maximum Dynamic Amplification Factor with Span

Note : ● and ● points corresponds to critical distances given in table 4.4

From the figure, it is noted that maximum dynamic amplification factor for any span is constant for change in system parameters, viz., pier dia, height and weight of span which confirms the observation made above.

Some peaks are noted in Span v/s maximum dynamic amplification factor graph. To understand these, critical distances given in Table 4.4 have been marked in Fig 5.77. Peaks are observed for span length equal to inter-coach bogie distance (≈7 m) and for odd multiple of half coach length. It is noted that these peaks corresponds to spans for which dynamic component was high (Fig. 4.26). It is also noted that dynamic amplification is nearly unity when span length is in multiples of one coach length.

5.6 CONCLUSION

In the present Chapter, dynamic analysis of piers of curved bridges for different configurations has been carried out. To summarise, following conclusions have been drawn from this study:

i) The dynamic effects of centrifugal force on pier are higher than that obtained from static analysis. Tendency of resonance is also noted in piers within operational speed of trains for many spans. Dynamic amplification in shorter spans (length less than one coach length) is higher than that in longer spans.

ii) For shorter spans (< 22m), dynamic centrifugal force is almost constant at lower speeds (upto 20 m/s) irrespective of span length and approximately equal to static centrifugal force for 22 m span. However, it is very high at higher speeds (30 m/s), e.g., for 10 m span, it is approximately double in comparison to static centrifugal force for 22 m span.

iii) For longer spans (> 22 m), the dynamic amplification is less in comparison to that for shorter spans. Maximum DAF for 32 m span has been noted to be 1.35 only against 3.68 for 6 m span.

iv) Maximum DAF occurs when span length is equal to inter-coach bogie distance (≈7 m) or in odd multiple of half coach length, viz., 1.5, 2.5 etc. The value of maximum DAF decreases with increase in span for such cases.

v) Minimum DAF (≈ unity) occurs when span length is in multiple of coach length, viz., 1, 2, 3 etc.

vi) Due to change in system parameters, viz., pier dia, height and weight of span; the speed at which maximum dynamic amplification occurs (i.e. resonance) changes, however, the value of maximum DAF remains unchanged. Hence, maximum dynamic amplification factor for any given span is constant.

CHAPTER-6

CONCLUSIONS AND RECOMMENDATIONS

6.1 PREAMBLE:

With the introduction of Metro Services passing through congested city areas, bridges with sharp curves have to be provided with high piers. Literature available on the subject has been reviewed. It has been noted that Extensive study on train induced ground vibrations has been carried out by various researchers; however, their scope has been limited to underground metro trains. In case of bridges, the scope has been limited to vibrations in deck due to resonance. Much literature is not available on centrifugal force in curved bridges except for codal provisions. It has been noted that though centrifugal force is essentially a dynamic phenomenon, it has been treated as static force in present codes.

6.2 PRESENT STUDY:

In the present study, elements of dynamic analysis in reference to bridges have been discussed and bridge parameters for proposed study have been identified (span Length, weight of span, height of pier, size of pier, speed of train). Methodology of dynamic analysis has been studied and explicit scheme for transient dynamic analysis has been discussed in detail.

Loading history required for dynamic analysis of piers with various spans has been generated for 8 coach metro trains consisting of Bombardier's MOVIA coaches. It has been noted that loading spectra varies from span to span. It has been observed that for span lengths close to multiples of coach length (22m and 44 m), loading spectra has negligible dynamic component. For other spans, it is observed to have significant dynamic component.

To understand the response of piers in curved bridges, dynamic analysis of bridges for various bridge configurations having 8-12 m high piers supporting spans ranging from 4-60 m on either side has been carried out for centrifugal force transferred due to passing of a Bombardier's MOVIA 8 coach metro train at speed varying from 5 m/s to 40 m/s. The calculated dynamic lateral load on pier due to centrifugal force has been compared with that obtained using present codal provisions and analysis of results carried out. Following conclusions have been drawn from this study:

i) Due to dynamic amplification, effects of centrifugal force on pier obtained from dynamic analysis are higher than that obtained from static analysis as per present codal provisions. Tendency of resonance is also noted in piers within operational speed of trains for many spans.

ii) On the basis of dynamic behaviour, the spans can be classified as either short or long span in reference to coach length. The spans less than one coach length (22 m in present study) can be classified as short spans.

iii) Dynamic amplification in short spans is higher than that in long spans.

iv) For short spans at lower speeds (upto 20 m/s), magnitude of dynamic centrifugal force on pier is almost constant irrespective of span and is approximately equal to that for 22 m span. However at higher speeds (30 m/s), dynamic forces are much higher. For 10 m span, magnitude of dynamic centrifugal force has been noted to be double in comparison to that for 22 m span. A dynamic amplification factor of 3.68 has been observed for 6 m span.

v) For longer spans, the dynamic amplification is less in comparison to that for short spans; however, its value is still significant. Maximum dynamic amplification factor for 32 m span has been noted to be 1.35.

vi) Due to change in system parameters, viz., pier dia, height and weight of span; the speed at which maximum dynamic amplification occur changes, however, the value of maximum DAF remains unchanged. Hence, for the given train, maximum dynamic amplification factor is a function of span only.

vii) Minimum DAF ($\approx$ 1.12) occurs when span length is in multiple of full coach length, viz., 1, 2, 3 coach length etc. Such spans (within $\pm$ 10% variation over multiple of full coach length) can safely be used irrespective of pier height, dia and weight of span with a DAF of 1.15.

viii) For longer spans in range of 1-2 coach length, high DAF ($\approx$ 1.35) occurs when length of span in order of 1.5 coach length. Such spans can also be used with a DAF of 1.40.

ix) For span in other odd multiple of half coach length, viz., 2.5, 3.5 coach length etc., the DAF is low ($\approx$ 1.12) and such spans can be used safely irrespective of pier height, dia and weight of span with a DAF of 1.15.

x) Maximum DAF (3.68) occurs when span length is less than one coach length. Use of such spans shall be avoided. If unavoidable, detailed dynamic analysis shall be carried out for such spans on case to case basis.

6.3 LIMITATIONS:

Following are the limitations of this study:

i) Only one type of coach (Bombardier's MOVIA coach being used in Delhi Metro) has been used in the study.

ii) Only 5% damping, which is usually recommended for RCC structures, has been used.

6.4 RECOMMENDATIONS:

On the basis of conclusions drawn in this study, recommendations are made on use of spans in curved bridges and given in Table 6.1.

Table 6.1 Recommendations on use of Spans in Curved Bridges

S.N.	Span	Recommendation
1.	Short spans: Less than 0.9 coach length	Better shall be avoided or Detailed dynamic analysis shall be carried out on case to case basis
2.	Long spans: 1.1 to 1.9 coach length	can be safely used with a DAF of 1.40 irrespective of pier height, dia and weight of span
3.	Other long spans: 0.9 to 1.1 and > 1.9 coach length	Can be safely used with a DAF of 1.15 irrespective of pier height, dia and weight of span

* * * * *

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