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Assessment of Plastic Deformation of the Soil in the Gujarat Region Caused by High-Speed Train

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Abstract: ***Over the last few years, a need for high speed locomotives for faster connectivity between cities in India has been recognized due to its superior economic, social and environmental benefits in comparison to other comparable means of transport. Given the background, safety and endurance become a vital need of the hour since it calls for a shift from conventional design, execution and maintenance approaches as regards High – speed locomotives. So, with an objective to ensure safety of the passengers, longevity of the infrastructure and minimize the impact of vibrations; we need to safe guard the mechanical components of trains and tracks from any such damage.***

Innovative approaches are required in the design and construction of strong railway roadbeds based on accurate modelling of the force induced and the corresponding impact on the track foundation caused by the high-speed trains at speeds exceeding 300 kmph as against the current conventional designs which cater to design speeds of up to – 180 kmph. In the captioned study plastic settlement of CL type of track foundation soil is computed at a point subjected to various magnitude of cyclic loading grounded on the method as put forward by Li and Selig

Index Terms: quarter car model, dynamic amplification, plastic strain, clayey soil, cyclic loading.

I. INTRODUCTION

Maharashtra and Gujarat are one among the most technologically advanced and affluent states in India with significant traffic of goods and passengers between them. However, the existing modes of transportation fall short in meeting such surging demand resulting in economic loss, loss of productivity and greenhouse emissions. The Indian Railways, Government of India, along with Japan has proposed the "Diamond Quadrilateral", the high-speed rails (HSR) network that will link the four major cities of India - Delhi, Mumbai, Kolkata and Chennai [9]. These are mainly high-density corridors and ranges from 135 to 991km in length. Given the background, safety and endurance become a vital need of the hour since it calls for a shift from the conventional design, execution and maintenance approaches to those relevant to high – speed locomotives. So, with an objective to ensure safety of the passengers,

The longevity of the infrastructure and minimizing the impact of vibrations on the mechanical components of trains and tracks, there is a need for an overhaul of the conventional approaches based on in depth researches and data modelling techniques. For the safe design and construction of strong railway roadbeds the innovative approaches are required that are based on accurate modelling of the forces generated and the corresponding impact on the track foundation caused by the high-speed trains at speeds exceeding 300 kmph as against the current conventional designs which cater to design speeds of up to – 180 kmph. Quite a few novel experimental studies, semi-analytical approaches and numerical concepts have been developed in recent years for the analysis of vibrations induced by high speed locomotives. Wang et. al. (2018) [5] investigated the dynamic response of the subgrade under saturated and unsaturated state. The fatigue test of the waterproof functional layer was also conducted to check its performance. The results showed that along with the depth of broken stone surface layer the vibration velocity increases and it decreases linearly with the increase in the depth from the base of the waterproof functional layer. Stolarik et. al. (2017) [6] made experimental measurements to study the dynamic influence of rail transport on various types of geotechnical structures. The experimental research was backed by the theoretical research. Their study specifically focuses on the dynamic response of the secondary lining as the train passes through a side tunnel tube. The study also covers the response of the reinforced concrete angled wall loaded with dynamic effects of rail traffic. Chiou et al. (2016) [3] combined 2.5D FEM model and thin-layer element to study the response of the 3D ground under the influence of series of vertical point loads. These series of moving loads replicate the motion of train running along a track. The dynamic response was captured at sub-critical speed, critical speed and super critical speed of train. Their study covers three different types of top soil layer. Bian et al. (2015) [2] coupled 2.5Dfinite element model and thin layer element to investigated the vibration response of the track structure and the surrounding ground.

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The dynamic response was recorded for the running train under the influence of four different track irregularities. The four typical wavelengths considered are 1.4m, 3.3m, 6.5m and 20.0m. The track irregularities were modeled in the form of cosine waveform and the train loading was modeled in the sine waveform. The wavelength representing track irregularities were carefully selected from the various field study done in China. Huang et al. (2015) [4] studied vibration characteristics of the low subgrade ballastless track and suggested use of pile-plank support to enhance the performance of poor subgrade. Later vibration response and long-term performance of pile-plank-supported low subgrade concluded that under variable frequency test, the proposed system to support low subgrade, exhibited smoother stiffness along the longitudinal direction of the subgrade. This will result in the effective control on the dynamic stiffness of the subgrade under the effect of variable train speed. Thus, will ensure driving safety. Jian-ganga et al (2013) [8] using ANSYS did dynamic analysis of the track on the transition zone from roadbed to bridge. For this analysis they developed discrete model to replicate track system which comprises of the track passing on roadbed-bridge transition zone. Using FEM technique, the vibrational displacement, rotation angle and surfacial stresses generated in the roadbed at the intersection with the bridge section were investigated for different length of roadbed-bridge combinations. The study is extended to comprehend the influence of different stiffness ratio and train velocity on the dynamic performance of the system. In the current study plastic and elastic deformation of the CL type of soil is studied under loaded and unloaded condition based on Li and Selig's method and theory of elasticity respectively.

II. MODELLING MECHANISM

The vehicle system model used in the current study idealizes CRH3-type high speed trains that runs on most high-speed corridors in China. The train body comprises of 10 or more carriages. The carriage is modelled as quarter car supported on two bogies and four wheelsets as illustrated in Figure 1(a) below. The body of a car and the axles are considered as rigid parts while the primary and secondary suspensions are represented as spring and damper. The load transmitted to the track depends on the car mass, $\bar{M}_{\mathrm{c}}$, the bogie mass, $\bar{M}_{\mathrm{b}}$, the mass of the wheels $\bar{M}_{\mathrm{w}}$, , the primary suspension, $\bar{k}_1$andc$_1$, the secondary suspension $\bar{k}_2$ and $\bar{c}_2$ and train configuration, $\ddot{x}c$, $\ddot{x}b\ddot{x}w$ and $u_c$represent vertical displacement of carriage, bogie wheel and track respectively. $\bar{k}_{\mathrm{H}}$ represents hertzian spring constant placed in between wheel and track. Table 2 gives the details of CRH3 type train as given by Bian e.t al. (2014) [2]. Thus, a 4-DOF dynamic system is developed to represent the vehicle model, equation 1. The dynamic equilibrium of equations representing motion of the vehicle are derived by not considering the influence of track irregularities. As per equation 2, P(t) is defined as the total vertical contact force acting at the wheel and rail interface. It comprises of both the static vehicle load and the contact force. As per the specification of BS EN 1991-2:2003 EN 1991-2:2003(E) [1] the point force exerted by the high-speed load model (HSLM-A; A5) with 14 intermediate coaches at each contact point is 170kN. In the current study, computed contact force, P(t) is162kN, which acts on the top of UIC60 rails (IRS-T12) as shown in fig 1(a) with less than 5% error. Based on the actual cross-sectional geometrical parameters of the high-speed railway in China, a numerical model coupling solid elements and frame elements is established to compute the impact of train speed on the dynamic response of the supporting soil when train moves on the track with no irregularities. To illustrate the impact of train speed the track system is subjected to a series of unit amplitude sinusoidal excitations and the nonlinear time history analysis is performed on it. The track system considered for this analysis comprises of UIC60 type rail system of height 172 mm with the mass and the section of the rail as 60.34 kg/m and 7686 mm respectively. The rail and sleepers are modelled using frame section in SAP2000 ver20. The sleepers resting on track slab, are considered of high strength concrete with characteristic cylinder and cube strength of 80 N/mm^2 and 95 N/mm^2 respectively. At the rail seat the top width of the sleeper measures 220 mm whereas the bottom width is 270 mm. The sleepers are modelled as frame elements at center to center distance of 0.65 m. Rest of the track structure is represented by 8-noded solid elements with the parameters as referred by Bian et. al. (2016) [2]. The slab track system comprises of a concrete slab on a hydraulic sub-base. The concrete slab has Young's Modulus as 3.9x10^4 MPa. The hydraulic sub-base has Young's Modulus of 1.0x10^4 MPa and width same as concrete slab. The foundation with slope 1:2 consists of three structural layers; from top to bottom they are 0.4m thick roadbed, 2.3m thick subgrade and 3.3m thick embankment body as shown in fig 2. The subsoil underneath is composed of heterogeneous strata comprising of silty clay with mud, silty clay and silt with modulus of elasticity as 45 MPa, 55 MPa and 65 MPa respectively. The bottom of the subsoil has fixed boundary condition. The model is finely meshed to form proper connectivity between 2-D and 3-D elements.

III. VALIDATION OF FEM APPROACH

For the validation study with respect to HSLM-A model, the 3D FEM model developed was subjected to series of sinusoidal excitations representing 200 kmph, 256 kmph and 300 kmph running speed of train. Fig 3 compares the vertical vibrational velocities of the concrete base obtained from the numerical simulation with the field measured data at a speed of 200 kmph, 256 kmph and 300 kmph, taking the CRH3-type train as an illustration. It is observed that the proposed model holds good correlation with the measured field observations and hence validate the precision of quarter car model and 3D FEM model for further application.

IV. INTERPRETATION OF FREQUENCY RESPONSE CURVE

National High-Speed Rail Corporation Limited, India (NHSRCL) for the first time is going to execute the project of high-speed train corridor between Ahmedabad and Mumbai city [10].

This high-profile bullet train will be designed for an operating speed of 320 kmph to cover the distance of 526 km in two hours and fifty-eight minutes which presently is covered in seven to eight hours. The Mumbai-Ahmedabad corridor will have in all 12 stations. In the current study, the effect of high speed on the soils of South Gujarat which covers the region from Valsad, Surat and Bharuch are premeditated. The major reason for selecting Surat city as the study area is that, it lies midway along the proposed corridor and has major market for textile industries and diamond trading [10]. From the geotechnical aspect also, the region is critical, since from the past literature it is found that majority of the Surat region is composed of black cotton soil which has a tendency to expand as it absorbs water and shrinks as it is dewatered. Since this project is first of its kind in India sufficient comprehensive literature on the impact of train speed above 200kmph on the vibration response of the track foundation soil in Indian terrain is not available. In the current study the validated 3D FEM is reconstructed by changing the property of foundation soil below embankment to CL type as referred by Patel et al. (2012) [7] for Surat City. A plot of dynamic amplification vs frequency ratio for CL and SC (sandy clays) is shown in Figure 4(a). The plot is prepared for the damping ratio of 0.05, 0.2 and 0.7. From the graph it is evident that when both the soils were subjected to similar forcing frequency, the magnitude of dynamic amplification factor at resonance is much less than the peak. Hence, we can say that as dynamic amplification factor tends to zero it is unaffected by damping. Thus, the dynamic deformation of the soil below an embankment will be less than the static deformation, implying that response is controlled by the mass of the system.

P(t) (b)

Fig. 1(a) Quarter Car Model (b) Rail Bogie System [5]

$$\begin{bmatrix} \bar{M}c & 0 & 0 & 0 \\ 0 & \bar{M}b & 0 & 0 \\ 0 & 0 & \bar{M}w & 0 \\ 0 & 0 & 0 & 0 \end{bmatrix} \begin{Bmatrix} \ddot{x}c \\ \ddot{x}b \\ \ddot{x}w \\ \ddot{u}c \end{Bmatrix} + \begin{bmatrix} \bar{c}2 & -\bar{c}2 & 0 & 0 \\ -\bar{c}2 & \bar{c}1+\bar{c}2 & -\bar{c}1 & 0 \\ 0 & -\bar{c}1 & \bar{c}1 & 0 \\ 0 & 0 & 0 & 0 \end{bmatrix} \begin{Bmatrix} \dot{x}c \\ \dot{x}b \\ \dot{x}w \\ \dot{u}c \end{Bmatrix} +$$

$$\begin{bmatrix} \bar{k}2 & -\bar{k}2 & 0 & 0 \\ -\bar{k}2 & \bar{k}1+\bar{k}2 & \bar{k}1 & 0 \\ 0 & -\bar{k}1 & \bar{k}1+\bar{k}H & \bar{k}H \\ 0 & 0 & -\bar{k}H & \bar{k}H \end{bmatrix} \begin{Bmatrix} xc \\ xb \\ xw \\ uc \end{Bmatrix} = \begin{bmatrix} \bar{M}c \\ \bar{M}b \\ \bar{M}w \\ 0 \end{bmatrix} g + \begin{bmatrix} 0 \\ 0 \\ 0 \\ P'(t) \end{bmatrix}$$

where, P(t) = P’(t) + [0.5(Mc + Mb) + Mw] g (2)

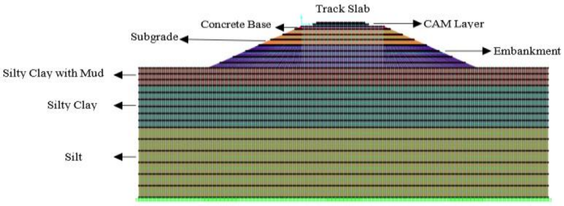


Fig. 2Representation of FEM of tracksubstructure used in the current study

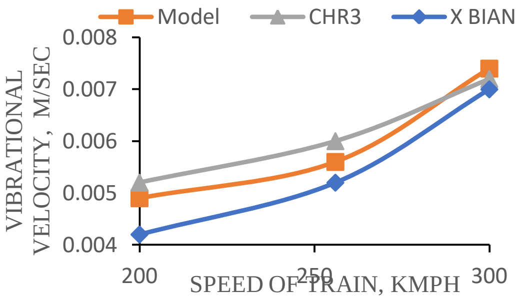
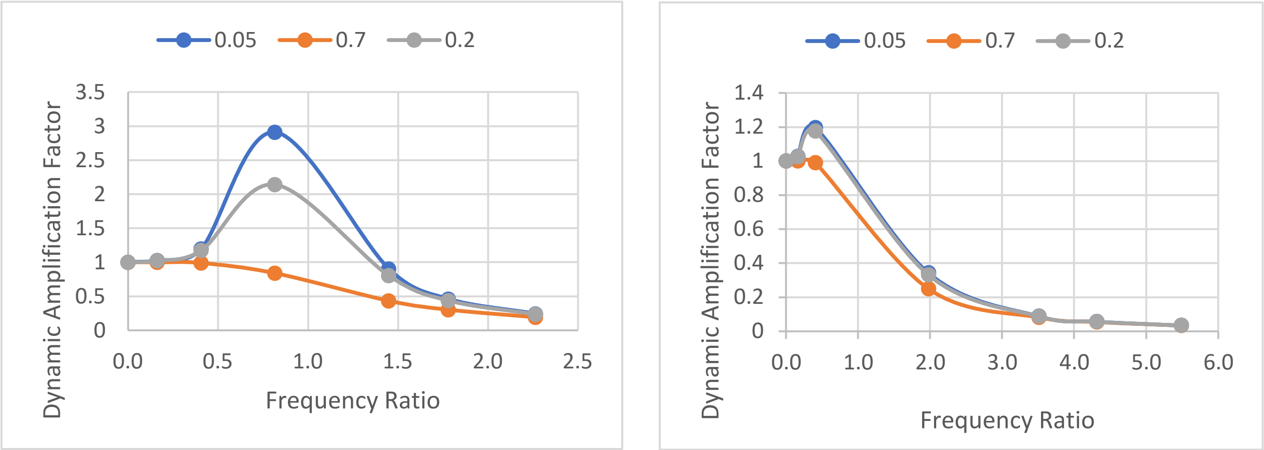


Fig. 3Comparison of vibrational velocity



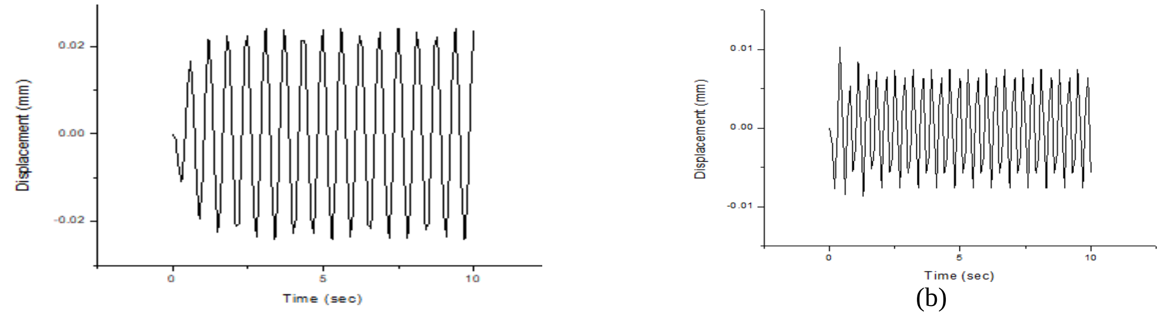
(a) (b)

Fig. 4 Frequency Response Curves for (a) SC type soil and (b) CL type soil

Table 1 Properties of soil used in study

Soil Type	Geotechnical Properties	Notations & Units	Values
CL	Density	ρ, kN/m^3	13
	Modulus of Elasticity	E, kN/m^2	2.9e03
	Poisson’s Ratio	μ	0.44
SC	Density	ρ, kN/m^3	14.7
	Modulus of Elasticity	E, kN/m^2	1.8e04
	Poisson’s Ratio	μ	0.44

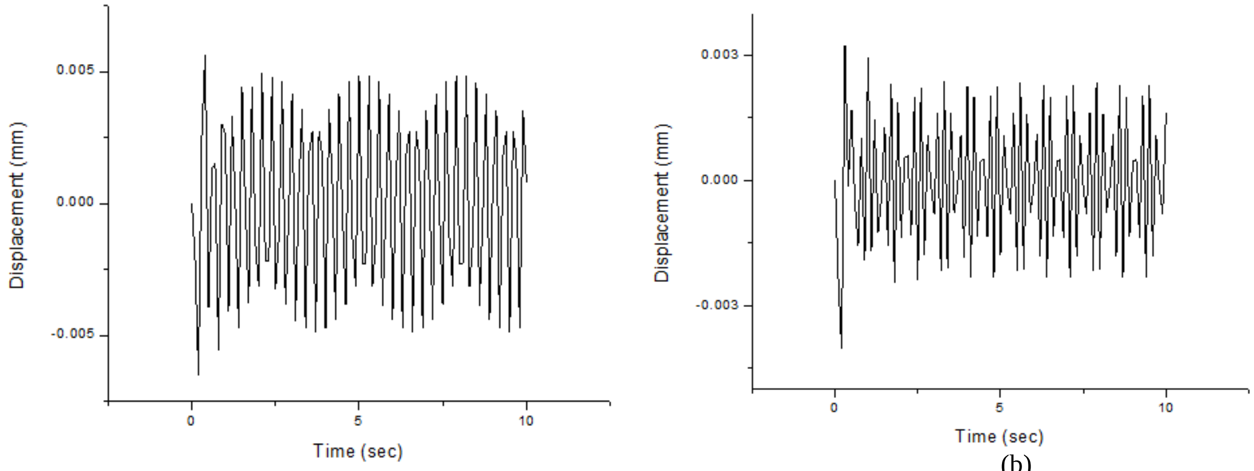
V. VIBRATION INDUCED DUE TO PROPOSED HST PASSING STUDY AREA



(a) (b)

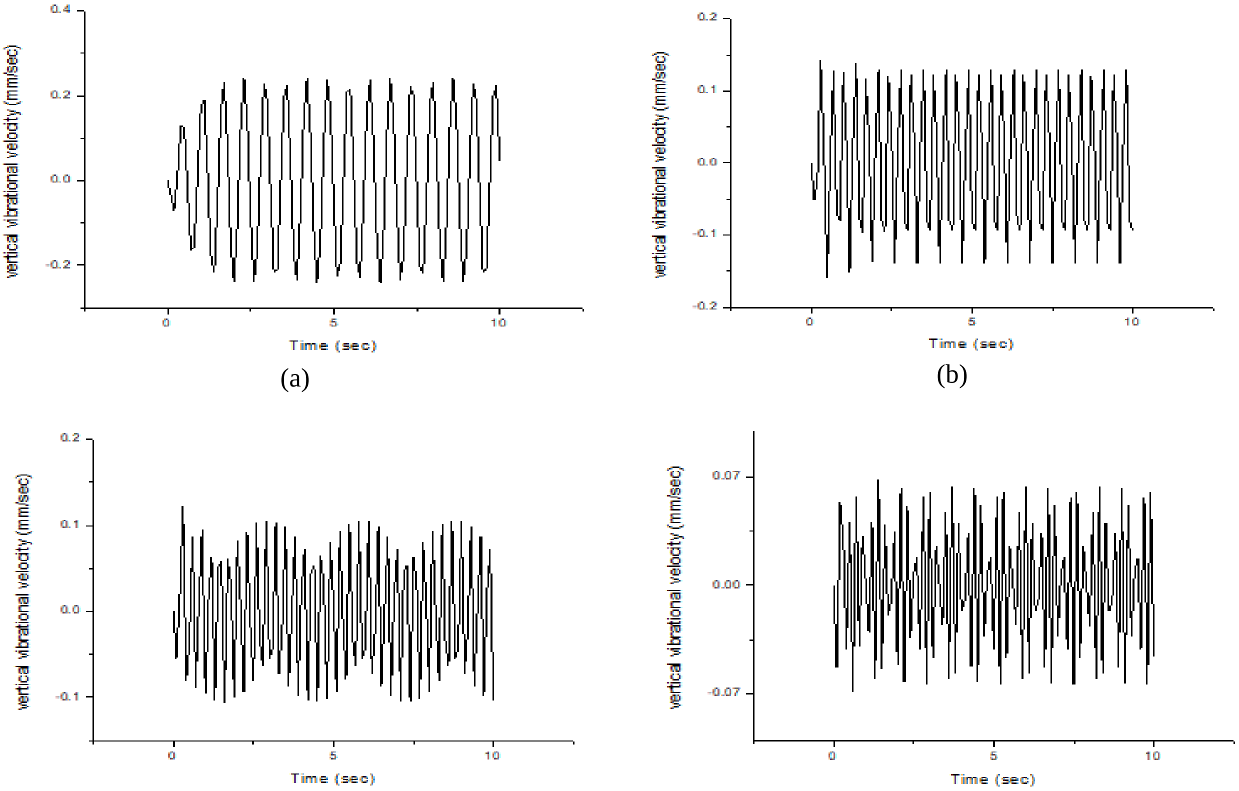
Fig. 5Time history of dynamic deformation in soil (a, b:v=144kmph, 256kmph)

A detailed investigation is done to simulate and evaluate the effect of repeated cyclic loading on the CL type foundation soil. The angular frequency of the sinusoidal load ranges from 0 to 40 rad/sec replicating train’s speed at 144 kmph, 256 kmph, 314 kmph and 400 kmph. The non-linear time history of vertical displacement and vertical vibrational velocity contain a series of periodic waveform. Figure 5 and Figure 6 represents wheel rail contact force response of the present model over 100 cycles of simulated train loading for examining the possible response in terms of deformation and vertical vibrational velocity in clay below embankment. From Fig 5(a and b) and Fig 6(a and b), the amplitude of dynamic displacement is measured to be 0.048 mm when Express. Figure 7 shows that based on simulation analysis maximum computed vibrational magnitude near study area is 0.21 mm/sec at a speed of 144 kmph and it further reduces the speed of train is 144 kmph; the displacement amplitude undermines to 0.02 mm at a train speed of 256 kmph, 0.012mm at a speed of 314 kmph and 0.0088 mm at a speed of 400 kmph, which is well within the permissible limits of 0.2 mm under dynamic loading. As per Supplemental Environmental Impact Assessment Report for Mumbai - Ahmedabad High Speed Railway Project submitted to NHSRCL on September 2018 [11] the existing ground borne vibration (peak particle velocity) at all locations along the line of proposed high-speed rail ranged between zero to 0.230 mm/sec. The highest vibration level was recorded at Ahmedabad, near the Railway track at 0.230 mm/sec during the passing of Rajdhani due to decrease in the dynamic amplification factor with the frequency ratio as explained earlier.



(a) (b)

Fig 6Time history of dynamic deformation in soil (a, b: v=314kmph, 400kmph)



(a) (b)

(c) (d)

Fig 7Vibrational velocity in soil (a, b, c, d: v=144kmph, 256kmph, 314kmph, 400kmph)

VI. STRESS-STRAIN BEHAVIOUR OF SOIL DUE TO INDUCED VIBRATIONS

Fine grained subgrade soils subjected to repeated train loading will cause permanent settlement due to cumulative plastic shear strain, cumulative consolidation and cumulative compaction (Li and Selig 1996) [2]. To understand the stress strain characteristics of the selected foundation soil the elastic and plastic deformation in the method is used to compute plastic deformation. Li and Selig in 1996 [2] proposed a method to compute the soil at various depth is studied under loaded and unloaded condition. The computation of the elastic deformation is done based on theory of elasticity and Li and Selig permanent settlement of fine-grained subgrade. Unloaded condition, described as settlement measured in the soil, in the absence of train load will be referred as DL, while settlement when measured in the presence of the train load (stationary) is loaded condition will be referred as DL+LL. Fig 8 (a) shows that plastic settlement increases with the increase in the depth also from the Fig 8(a) it can be deduced that with the increase in the loading there is no significant change in the elastic settlement. Fig 8(b) represent plastic settlement at 6m, 9.6m and 13.2 m for the DL+LL condition when the train is moving at the speed of 144, 256, 314 and 400 kmph. It is observed that the cumulative plastic strain converges to zero with the increase in the speed of the train from 144 kmph to 400 kmph in 400 secs. It can be said that the increased speed of the train caused dynamic compaction of the soil and hence leaving no scope for the further readjustment of the soil particles. As a result, the soil gets compacted and gain strength. From Fig 9(a) it is evident that after the passage of third carriage at a speed of 144 kmph the cyclic stress generated in the saturated soil mass is 2.98 N/mm^2 at a strain of 5.8e-04% and same after passage of 13th carriage at a speed of 144kmph decreased to 2.92 N/mm^2 at 5.7e-04%. For the third carriage of CRH3 type train running at speed of 314 kmph Fig 9(b), which will strike the same point after 300 secs, the cyclic stress generated at the same point is 2.44 N/mm^2 at 2.38e-04% cyclic strain and after passage of 13th carriage it is reduced to 2.42 N/mm^2 at 2.4e-06%. It is observed that the hysteresis losses in both the cases is negligible after 3rd carriage while little hysteretic loss is observed after passage of 13th carriage. The stress-strain response of the test soil shows that the induced loading up to 400secs results into only elastic compression of the soil, as the elastic deformation of the soil as seen from the Fig 9(c) and (d), occurs without any change in the volume of the soil mass, hence the soil when dynamically loaded is within its elastic limit.

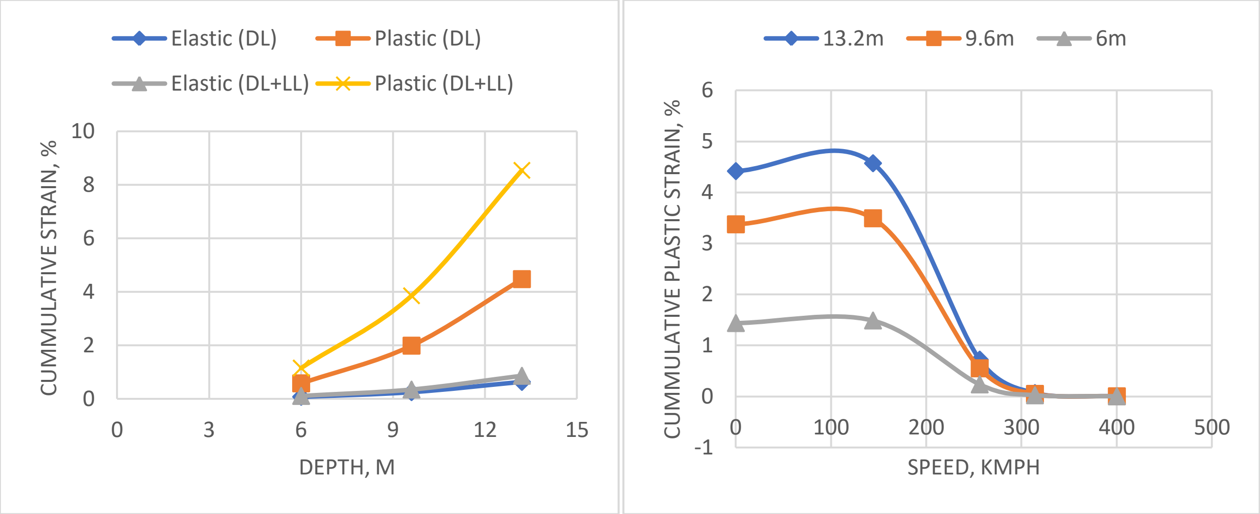
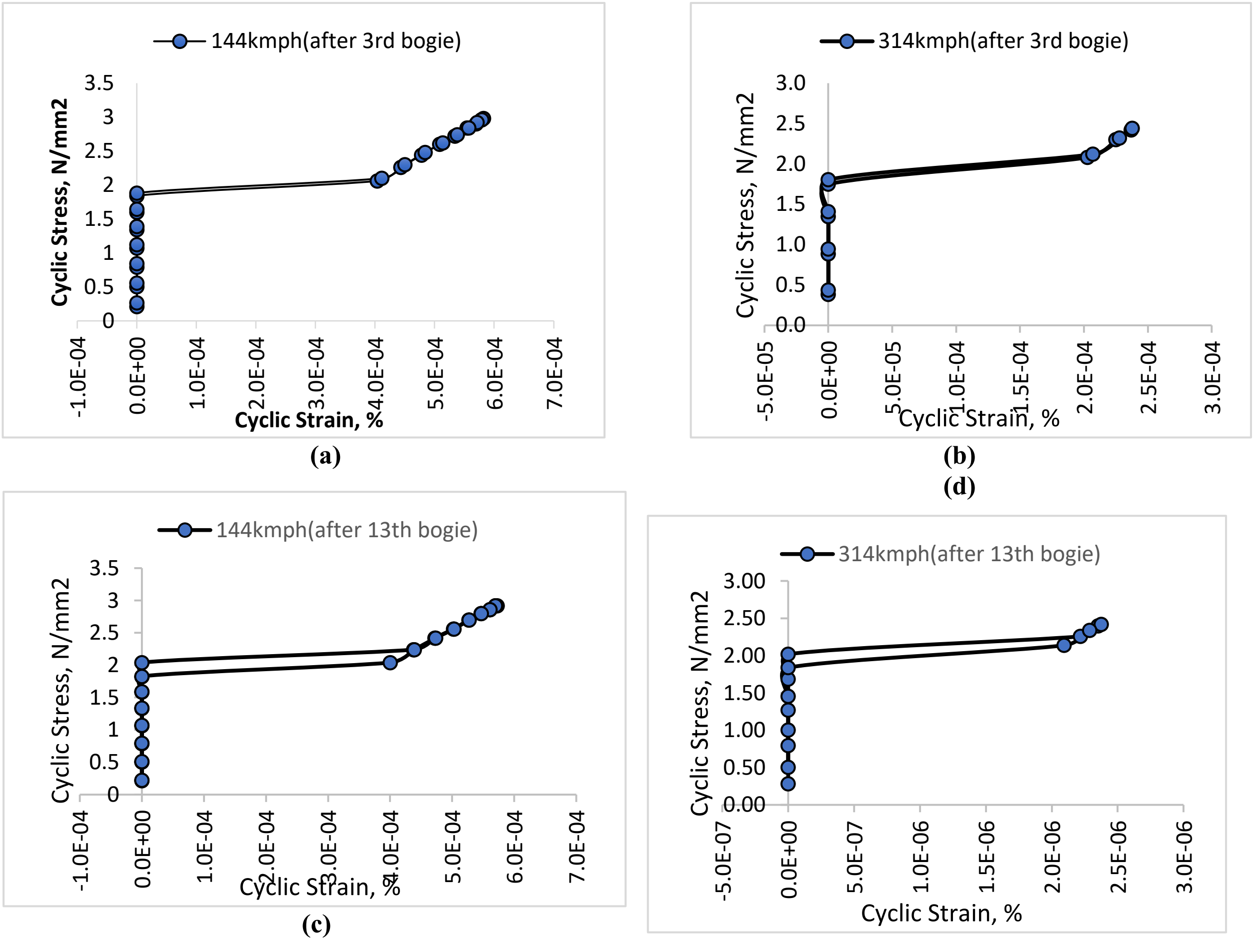


Fig.8(a) Plastic and elastic strain (loaded and unloaded condition) (b) Plastic strain (dynamic condition)

VI. CONCLUSION

Based on the study following conclusions can be drawn:

1. The vibrational velocity computed at various speed using proposed 3D FEM model holds good correlation with the existing CRH3 train's field observations. The magnitude of live load representing train load used for dynamic analysis derived using quarter car model is verified with the specification as mentioned in BS EN 1991-2:2003 EN 1991-2:2003(E) with the percentage error less than 5%.
2. From the frequency response curve for SC and CL type soil it is clear that for the similar type of track structure and dynamic loading the dynamic amplification in case of SC type soil will be nearly twice as compared to CL type soil.
3. Under the influence of static loading of the CRH3 type high speed train on CL type of soil, excessive cumulative plastic strain is observed with the increasing depth from the base of embankment, inferring that the particles readjusted themselves to achieve equilibrium under load. Further, when the soil is subjected to dynamic loading, with every increase in the frequency ratio, reduction factor tends to zero, implying that vibratory deformation due to rapidly varying load is very small. Thus, it can be also concluded that under the effect of dynamic loading not much variation in the stress and strain parameters is observed and the deformation is within elastic state deducing that no further noticeable rearrangement of particles is possible. Thus, the train load acted as a preload on the soil that finally caused consolidation of soil.



(a) **(b)**

(d)

(c)

Fig 9. Stress-strain characteristic of CL type soil after passage of train

Table 2: CRH3 Type High Speed Train Parameters (Bian et al) [1]

Name	Notation	Value	Unit
Mass of carriage	$\bar{M}_c$	40,000	kg
Mass of bogie	$\bar{M}_b$	3200	kg
Mass of wheel axle	$\bar{M}_w$	2400	kg
Stiffness of primary suspension	$\bar{k}_1$	2.08e06	N/m
Damping of primary suspension	$\bar{c}_1$	1.0e05	N/m
Stiffness of secondary suspension	$\bar{k}_2$	8.0e05	N/m
Damping of secondary suspension	$\bar{c}_2$	1.2e05	N/m
Distance of wheel axles in a bogie	a_n	2.5	m
Distance of bogies in a carriage	b_n	14.875	m
Length of carriage	L_n	25.0	m

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