Performance of primary tunnel support systems under seismic loads in weak rock masses

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***Abstract*—With the decrease in the over ground utilizable space and increase in growth of metro structures, cut and cover structures are becoming rather impossible to conceptualize and construct. Hence they are slowly replaced by underground openings made by NATM methods. These tunnels could be using rock bolts and shotcrete as support systems for poor-fair quality rock masses whereas ground improvement will be needed for extremely poor quality deposits. At this point it will be prudent enough to understand the seismic performance of these support systems if such a demand comes in short term. Since for extremely weak deposits grout curtain for ground improvement acts as a barrier, the focus will be on better quality rock masses where rock bolts and shotcrete are only used as primary supports. For the same a relatively low RMR value is adopted for the rock mass under consideration. A typical NATM tunnel geometry is adopted which has to be excavated using conventional methods. Suitable support systems for the same are designed. Then seismic performance of these support systems are deduced using two different approaches namely (1) Pseudo Static Method as per IS 1893 and (2) Quasi Static displacement approach as per EC-8. A sensitivity analyses for the support systems at different cover depths are explored. Comparative studies of the above approaches are made *and their applicability for similar problems is discussed*.**

Keywords—Pseudo Static Analysis; Quasi Static Analysis; Seismic Analysis.

I. INTRODUCTION

In congested cities, owing to the rising urban demands transportation facilities by metro network have become quite common. Under such environment, metro stations are normally constructed by cut and cover methods (either top down or bottom up). But in some cases areas above station foot print may not be accessible due to a variety of reasons like inhabitation, traffic, important utilities etc. Under such circumstances cut and cover excavation is made in the proximity with access being made to these stations by *NATM* methods. These conventional excavation methods will be using rock bolts and shotcrete as primary support systems (in poor-fair quality rock masses) followed by a reinforced concrete liner with a water proofing membrane sandwiched in between. Since these excavations are made at shallow depths, it will be appropriate to understand the seismic demand of these structures. Such seismic demands are generally assumed to arise in "long term" i.e. once when these primary supports are degraded. Hence in such circumstances the permanent liner is overdesigned accounting seismic demands ignoring effect of primary supports (incorporating load factors depending on the design approach used). But for less corrosive environment, in short term condition, some seismic demand could be assumed to be shared by primary supports resulting in economical sections for the final liner.

In this study two most important methods to appreciate seismic demand (for underground structures) used in industry practice will be investigated. They are (1) Pseudo static analysis as per IS 1893 (2) [11] Quasi Static analysis as per EC-8. The former one is an inertial method used primarily for surface structures incorporating some recommendations made for underground structures.

The latter one is a displacement method in which certain prescribed displacements will be applied on these structures to find out their corresponding response. Both these methods will be used for a typical conventional tunnel, in a typical homogenous geotechnical profile to come up with suitable recommendations so that it could be furthered to conventional practice.

II. GROUND PROFILE AND TUNNEL DIMENSIONS

A. Ground Profile

For this study, for easy comprehension, homogenous rock mass is considered. At shallow depths owing to stratification and residual nature of soil/rock, poor-fair quality rocks are expected to be encountered. Thus a rock mass in accord to poor quality rock mass with *RMR* value of 35 ('*R1*' Grade [4], is considered (expected to have a stand up time of 10 hours for 2.5m span).

This rock mass is assumed to be highly weathered or altered with an intact uniaxial compressive strength (σ_{ci}) of 7MPa. Based on these two values (relatively conservative for rocks) all other design parameters for tunnels are arrived at.

The significant parameters to be calculated at this stage are Tunnel quality Index (Q), Mohr Coulomb parameters (c and ϕ), Joint roughness coefficient (J_r) and Geological Strength Index (GSI). Joint roughness coefficient (J_r) is taken as 1.0 [1], which is in agreement with smooth and planar joints. Following relationships will be used for calculation of the outstanding parameters.

$$\text{For GSI} > 25;\ \text{GSI} = \text{RMR-5} \tag{1}$$

$$\text{RMR} = 13.5 \log Q + 43 \tag{2}$$

For the calculation of equivalent Mohr coulomb parameters, Modulus ratio (*MR*), Disturbance factors (*D*), constant '*mi*' are required (Hoek, 2001). Modulus ratio, *MR* is taken as 350 and '*mi*' is taken as 25 (rocks of basaltic origin). No disturbance is assumed to rock mass during excavation (Mechanical hand excavation in poor quality rock). Based on these parameters, for uniaxial compressive strength (σ_{ci}) of 7MPa, *M-C* parameters are arrived at. After calculation when equivalent Mohr coulomb parameters are arrived (Hoek,2001), a cohesion (*c*) of 0.339 MPa and friction angle (ϕ) of 32 degree are obtained.A deformation modulus of 199MPa is obtained [3], for the rock mass. Poisson's ratio of 0.2 is assumed [6], for rock mass (basaltic rocks). After the calculation of geotechnical parameters, tunnel dimensions have to be specified.

B. Tunnel Dimensions

The tunnel under consideration will be constructed by conventional, *NATM* methods. The following tunnel will be excavated by heading and benching method. Typical tunnel profile with head and bench is shown below " Fig.1. Tunnel Excavation Dimensions

The tunnel as explained will be constructed using conventional methods i.e. heading and benching excavation sequence. The *NATM* excavation considered here would have a height (maximum height from invert to crown) of 9.83m and maximum width (at springing line) of 11.15m. This will be excavated as two segments i.e. a heading height of 6.83m and benching height of 3m. At first the excavation will be performed at the heading up to round length which is further followed by benching excavation. The magnitude of round length will be calculated based on Tunnel Quality Index (*Q*) in the design stage [7].

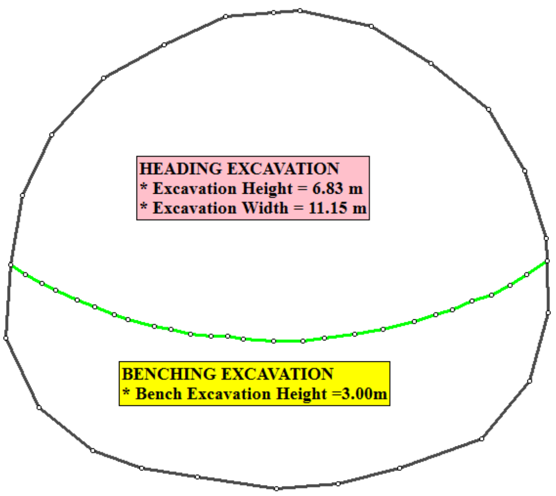

Fig.1. Tunnel Excavation Dimensions

III. Calculation of Primary Support Systems

For the calculation of primary support systems (rock bolts and fiber reinforced primary liner) initial proportioning was done based on Tunnel Quality Index (*Q*), Span or Height of Tunnel (whichever is higher) and Excavation Support ratio (ESR) [2]. For *RMR* value of 35, *Q* value of 0.3 could be arrived at as per(2). *ESR* value [7], of 1.0 is adopted (for major road and railway tunnels).

For this '*Q*' value of 0.3 and '*ESR*' value of 1.0, 1.23m unsupported length (round length[7], could be anticipated, which is sufficient enough to raise Lattice girders/I sections.Bolt Length (*L*) for this rock mass could be also calculated as per recommendations by [7], and 5m is found to be satisfactory. Primary support system required, in terms of primary liner (shotcrete or SFRS) and spacing of bolts are calculated as per [2]. It has been found that a bolt spacing of 1.5m c/c is adequate in shot created area. No bolting is performed at the invert as appreciable "heave" is not expected. Steel Fiber reinforced shotcrete (*SFRS*) 150 mm thick is used as the primary liner.

Once preliminary proportioning of liner and bolt system are performed, now it has to be checked against Indian standards (IS 15026, Tunneling methods in rock masses – guidelines[8]. The fundamental principle is to calculate support pressure requirements against an empirically calculated rock loading pressure 'p_v' [7]. For a '*Q*' value of 0.3 and 'J_r' value of 1.0, rock loading pressure is estimated to be 0.298MPa. *SFRS*, which is expected to fail solely by shearing, is estimated to have a capacity of 0.227 MPa (As per IS 15026, Clause 7.8.1). For the calculation of *SFRS* capacity 150mm thick liner is assumed having shear strength of 550t/m^2. Since the rock loading pressure exceeds resisting pressure, additional supports by Lattice girders are required. For remaining support pressure lattice girder pantex (*P70-20-25 @ 0.9m c/c*) is sufficient for a factor of safety 1.5. Contribution of bolts is ignored in support capacity calculation (as function of these bolts is solely stabilization of loose rock blocks around tunnel periphery). To sum up *SFRS* (150mm thick) reinforced with Lattice girder pantex (*P70-20-25* @ 0.9m c/c) with swellex bolts (5m long) is assumed as support system.

Before proceeding to finite element analysis, it will be appropriate to conduct a rock support interaction analysis [5], to confirm support systems derived by empirical methods. Here again contribution of rock bolts in capacity is ignored as these systems are only used as a "stitching" system for the destabilized rock mass. This analysis is performed at the maximum cover depth (i.e. maximum virgin stress). Shotcrete (150mm thick) is used as the main component of primary support. Ground reaction curve is plotted and Factor of safety of support system is calculated. For the rock mass,*UCS* value of 7MPa, *GSI* value of 30 and *mi* value of 25are used.As these methods are applicable only for circular sections in hydrostatic field, excavated area of the opening, is used to calculate the equivalent diameter (D_{eq}). From the excavated area equivalent diameter is 10.7 m. It has been found thatwith no support installed radius of plastic zone (r_p) extends to 10.36 m (from tunnel centre)i.e.

5 m from excavated face. The bolts are already extended up to 5m (from excavated face) as per empirical methods. With support systems installed, the wall displacement reduces from 0.81% to 0.3% of the excavated diameter. At the support location, 1.25m from excavated face, convergence is about 0.3 % of tunnel diameter. At tunnel face a convergence value of 0.25% is obtained.Since convergence confinement method is having some inherent assumptions, the result obtained from this analysis could be only considered as an indicative. Also systematic excavation, heading and benching could not be simulated using this method. Longitudinal displacement profile [9], is taken. Ground reaction curve and support plots are given below Fig.2.

The radius of plastic zone has reduced from 10.36m (5m from tunnel periphery) to 6.95m (1.61m from tunnel periphery). Anyways rock bolts are kept at 5m itself as systematic driving may be required at unsupported spans.

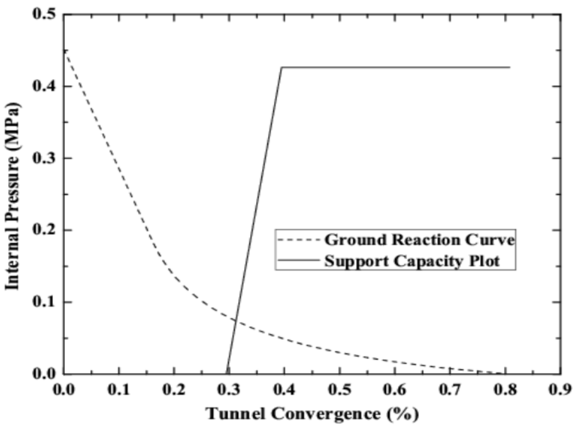

Fig.2. Convergence Confinement Curve & Support Plot

The major pitfall of rock support interaction analysis for such a problem could be

1) Inherent limitations of the above analysis
2) Support systems are used in seismic zones.

Thus, as deduced by empirical approaches, 150mm *SFRS* with Lattice girder pantex (P70- 20-25 @ 0.9m c/c) with 5m bolt length is used as primary support system. Now *FE*analysis for these systems needs to be performed at different cover depths.

IV. Finite Element Analysis of Support Systems

It is known that for any excavation, normal thrust on the tunnel liner will increase with cover depth. Hence as far numerical parametric analysis is concerned, 'cover depth' could be considered as a main parameter. For this analysis, minimum cover depth considered is 6.5m (1.5m is the clearance from bolt tip to finished ground level). Further this cover depth is incremented at 5m intervals. This means a cover depth of 6.5m, 11.5m, 16.5m and 21.5m are considered.

For FE analysis, sequential excavation stages could be simulated in Finite element software, *PLAXIS 2D*. The construction sequence of *NATM* tunnel is simulated (numerically) through following steps:

1) Geostatic Condition (Virgin stresses generation and nullifying the residual displacements at the end).
2) Stress relaxation at the heading portion to simulate three dimensional arching at the heading.
3) Applying shotcrete & bolting at the heading portion allowing stresses to load primary supports.
4) Repeating steps (2) and (3) in a sequential manner at the bench.
5) Performing dynamic analysis by Quasi static or Pseudo static approaches.

Reinforced permanent concrete liner installation or deactivation of primary supports is not considered in the analyses, since the intention of this paper is to understand performance of primary supports during seismic condition. The Staged excavation sequence performed during *FE* analysis is shown below "*Fig.3*".

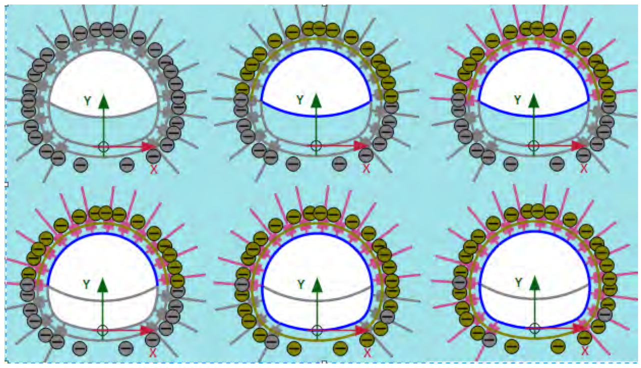

Fig.3. Sequential Excavation Process for FE analyses

Main challenge during these analysesis the calculation of appropriate stress relaxation coefficient to be used. As explained in rock support interaction analysis wall displacements could be used as an indicative. Since *FE* analysis could simulate 'K_0' condition, crown displacements for the deepest tunnel (cover depth =21.5m) is calculated at different relaxation coefficients. For a stress relaxation of 75% at the heading, crown displacement "*Fig.5*" was obtained as 28mm (i.e. less than 0.3% of tunnel diameter).This relaxation factor is considered same for all cover depths since it is an indicative of round length (generally a constant value).

Another important observation of this analysis could be surface settlements. The computed surface settlementsat different cover depths are given below "*Fig.4*". It could be seen that with an increasing tunnel cover depth, the maximum surface settlements were found to decrement [10]. Some significant deductions could be arrived from sequential excavation analysis. As expected with increase in cover depth bolt forces, moments and normal thrust were found to increase (due to confinement "Fig.6". Since the primary supports are designed for maximum forces, the focus will be on these absolute maximums. As far as bolt

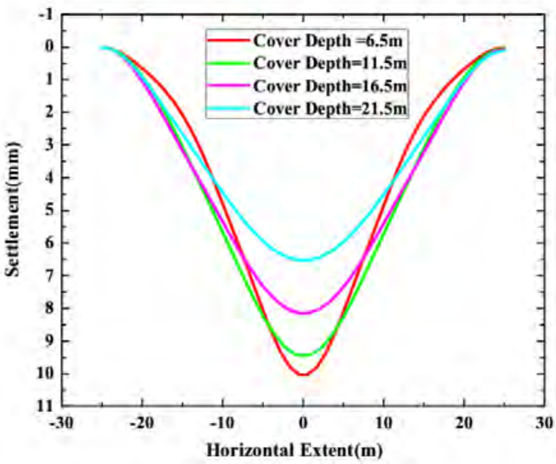

Fig.4. Surface Settlement Profiles at Different Cover Depths

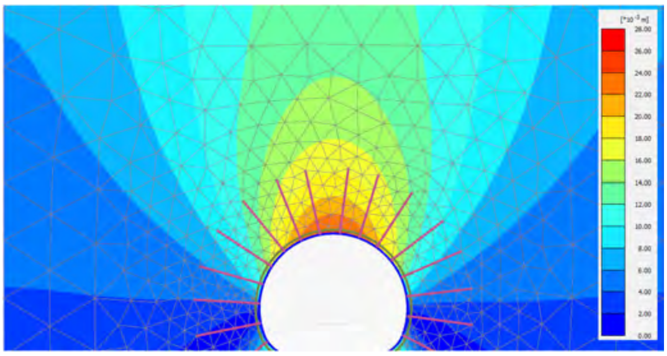

Fig.5. Crown Displacements for the Deepest Excavation

forces are concerned, the maxima are experienced on the tunnel walls Fig.7.

As far as normal thrust on the tunnel liner is concerned the maxima is experienced at the crown "Fig.7". Due to low stiffness of shotcrete liner moments are quite low to be mentioned.

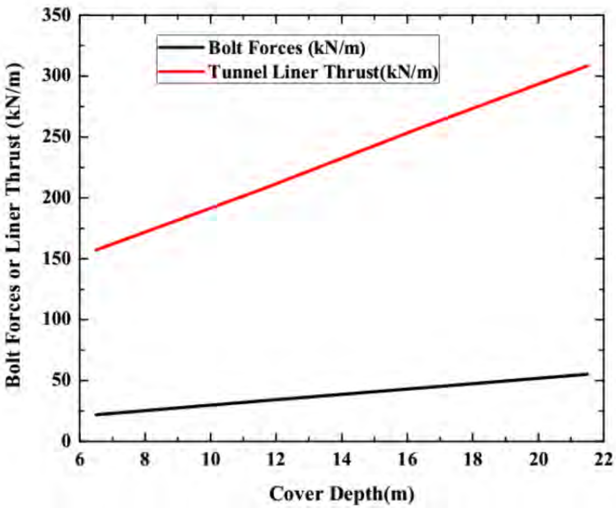

Fig.6. Variation of Bolt Forces and Liner Thrust with Confinement

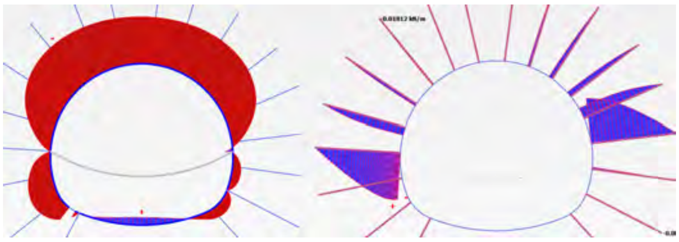

Fig.7. Axial Thrust and Bolt Forces Profiles on Primary Supports

V. Conventional Seismic Design Approaches

In conventional practice two different methods are used to appreciate the effect of seismic forces on underground structures. Full dynamic analysis (using earthquake time history) is generally less performed due to the absence of site specific dynamic parameters, computational resources and solving times. Hence the two major methods which are normally adopted for seismic forces computation are,
(1) Pseudo Static analysis (inertial method) using *IS 1893*
(2) Quasi Static analysis (displacement method) using *EC-8*.
Both these analysis are performed as a 'final' stage in numerical analysis to simulate seismic scenario. In pseudo static analysis design horizontal seismic acceleration coefficient 'A_h' (as per Clause 6.4.2 of *IS 1893*) is calculated. Further for underground structures, these values are reduced by certain fractions (the effect is reduced linearly from the surface down by 50% at 30m depth, Clause 6.4.4 of *IS 1893*). Now for this problem it has been observed that maximum liner/bolt forces will be obtained for quake motion in the downward direction (acceleration coefficients downward).In India, three major seismic zones III, IV and V are there.The pseudo-static coefficients used for this analysis for separate models (at different cover depths-for all zones) are given below (Table. 1). Vertical acceleration coefficients are taken as 2/3rd of the horizontal seismic coefficients.

TABLE I.Seismic Coefficient for Pseudo Static Analysis

Cover Depth(m)	*A_h(Zone 3)*	*A_h(Zone 4)*	*A_h(Zone 4)*
6.5	0.0911	0.1215	0.1822
11.5	0.0817	0.1090	0.1634
16.5	0.0723	0.0965	0.1447
21.5	0.0630	0.0840	0.1259

For Quasi Static analysis (displacement analysis), certain parameters have to be defined first hand. Subsoil may be classified as *Type A* based on average shear wave velocity for top 30m depth (V_{s30}=800m/s). No significant site amplification is expected for this soil type from bed rock (Soil factor, S =1.0). For tunnels, at different cover depths certain reduction ratio (C) has to be imparted for these tunnels (0.9 for 6-15m deep tunnels and 0.8 for 15-30m deep tunnels). Ratio of peak ground velocity to peak ground acceleration is given by Power et al.1996. For this specific problem a value of 109 may be taken as this value, for a source to site distance of 20-50km (in rocks) for an earthquake magnitude, M_w =7.5. Peak ground velocity (C_s) ranges from 1-5km/s. Thus a mean value of 2.5km/s is assumed. For all the finite element models cover depths and surface displacements (δ_x in mm) compatible with soil strains for different zones have been listed below (*Table*).

TABLE II.LATERAL TRANSLATION VALUES FOR QUASI STATIC ANALYSES

Cover Depth(m)	*δx(Zone 3)*	*δx(Zone 4)*	*δx(Zone 4)*
6.5	18.488	24.650	36.975
11.5	26.561	35.415	53.122
16.5	30.786	41.048	61.572
21.5	37.962	50.616	75.924

But before imparting these displacements it will be appropriate to confirm whether finite element method using *PLAXIS 2D* produce relevant results. Hence validation of the same with a classical published problem is required.

VI. VALIDATION OF QUASI STATIC SOLUTIONS USING *FE* ANALYSIS

To confirm the validation of quasi static solutions for excavation problems, [11], solutions for No –Slip conditions in tunnels are used after recommendations by [12]. As per this literature, tunnel liner modulus, *E* is taken as 24800000kN/m^2. Subsequently other parameters used in the literature are liner thickness (*t*) =0.3m, maximum shear strain at surface (γ_{max})=0.252% and model height =18m. The solutions proposed by [11] does not account for stress relaxation. Hence such a case is simulated in *PLAXIS 2D* and validated closed form solutions. To account the mentioned maximum strain value at the surface a horizontal displacement of 45.36mm is imposed. Soil mass as mentioned in [12], is assumed to have a modulus of 312 MPa and Poisson's ratio of 0.3. The *PLAXIS FE* model in which lateral translation is imposed is shown in "*Fig.8*". It could be seen that fairly good comparison "*Fig.9*&10", could be seen for *FE* solutions and [11] closed form solutions for classical published problem [12]. Hence Plaxis 2D performing FE analysis could be used for Quasi Static Problems.

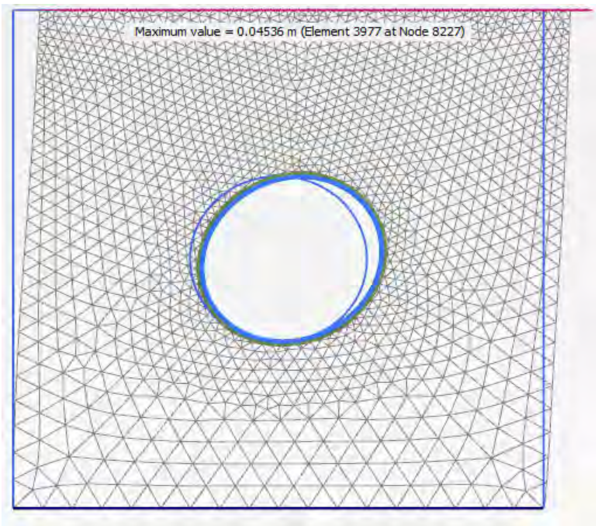

Fig.8. Lateral translation as per Hashash et al. 2005 (45mm)

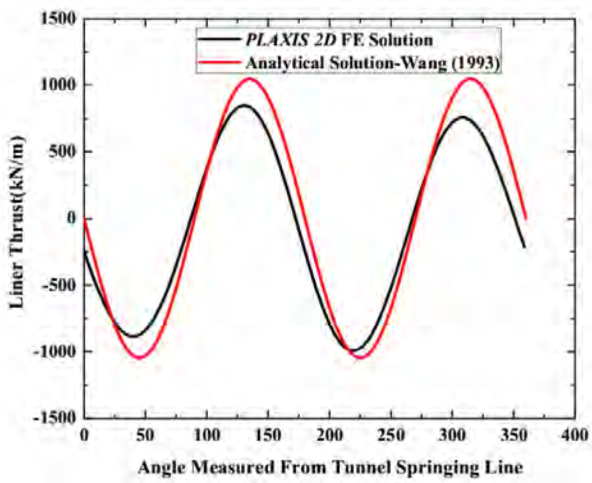

Fig.9. Tunnel Liner Thrust Comparison

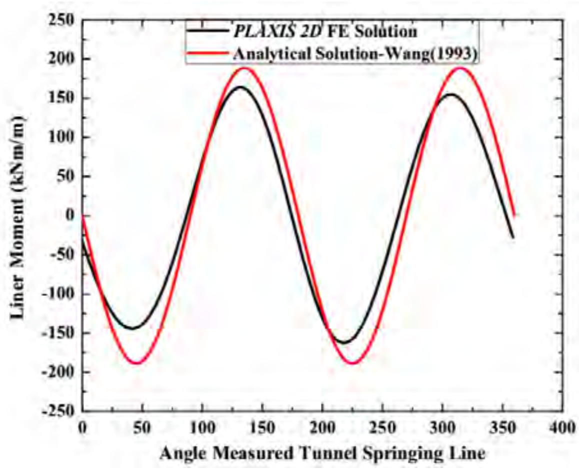

Fig.10. Tunnel Liner Moment Comparison

VII. FORCES EXPERIENCED ON THE PRIMARY SUPPORTS (PSEUDO VS QUASI ANALYSIS)

A. Bolt Forces

In Pseudo static analysis, zonal earthquake coefficients (as per *IS 1893*) are in increasing order from *Zone III* to *Zone V*. Thus it is expected that bolt force will increase from *Zone III* to *Zone V* at a particular cover depth. However with increase in cover depth the influence of earthquake coefficients tend to diminish at a rate mentioned in *IS 1893*, Clause 6.4.4. Thus it is expected that the rate of increase of bolt forces will decrease with increase in cover depth from *Zone III* to *Zone V*. To verify these observations zonal coefficients are plotted as abscissa and bolt forces as ordinates at different cover depths "*Fig.11*". It could be observed that the *FE* analysis conducted in *PLAXIS 2D* agrees with both the expectations.

In Quasi Static analysis, again the bolt forces are expected to increment from *Zone III* to *Zone V* as the induced displacements are at an increasing rate. Also like Pseudo static analysis the rate of increase in bolt forces will decrease with increase in cover depth from *Zone III* to *Zone*

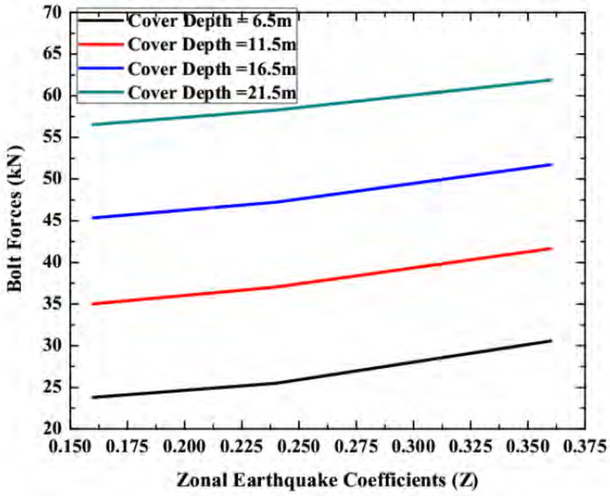

Fig.11. Variation of Bolt Forces with Zonal coefficients for different Cover Depths - Pseudo Static Analysis.

V. The important observation could be the fact that, this rate of increase in bolt forces are at pronounced rates at low cover depths. This could be attributed to the fact that due to linearly increasing displacements from the model base the soil elements will cause more "pull" on the bolts during ovalling. This appalling rate of increase in bolt forces and its dependency with cover depth could be seen in "*Fig.12*". It could be also seen that in Pseudo analysis maximum axial pull is experienced for bolts at tunnel walls. However in Quasi static analysis apart from tunnel walls some pull is experienced by bolts at crown. In the former case it could be due to the presence of horizontal seismic coefficients. In the latter case it could be due to lateral translations experienced above tunnel crown which could apparently create some pull "*Fig.13*".

Thus it could be concluded that application of Quasi static analysis for low cover depths for high earthquake zones could result in very high bolt forces in comparison with ones observed during pseudo static analysis.

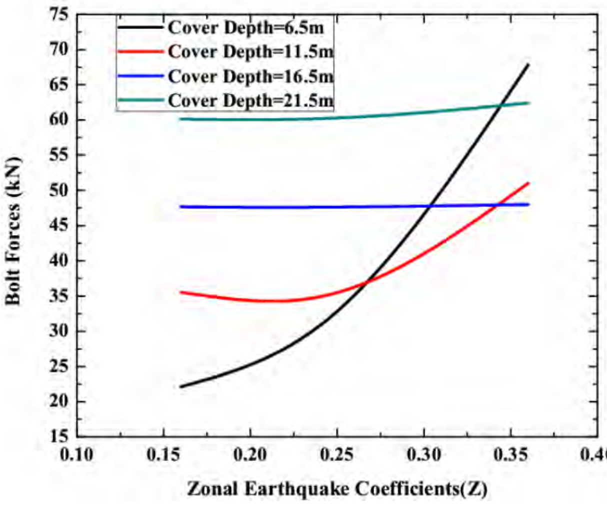

Fig.12. Variation of Bolt Forces with Zonal coefficients and Cover Depth- Quasi Static Analysis

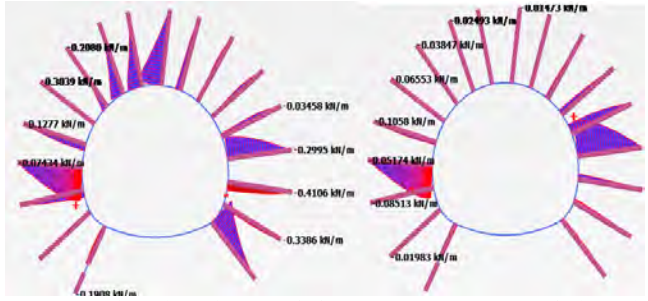

Fig.13. Location of Maximum Bolt forces (1) Quasi Static Analysis (Left) (2). Pseudo Static Analysis (Right)

In pseudo static analysis, maximum bolt forces are experienced at a cover depth of 21.5m (high confinement) in *Zone V* areas (61.90 KN). However in Quasi static analysis maximum bolt forces were experienced at a cover depth of 6.5m (due to high lateral translation effect) in *Zone V* areas (67.82kN). For these values 24mm bolts of yield strength (f_y)=415 MPa with a Safety factor 2 could be deemed satisfactory.

B. Liner Forces

The major liner forces that should be considered are axial thrust and moments experienced on the liner. Moments will be significantly low due to the lesser stiffness of the liner elements. In Pseudo and quasi static analysis the thrust values were found to increase with an increase in zonal earthquake coefficients at low cover depths (Cover depth = 6.5m and 11.5m). However at higher cover depths, the axial thrust were not much affected (in this case decrease) with zonal coefficients. This could be attributed to the load sharing phenomena between liners and surrounding soil elements at high confinement. Thus as far as Pseudo static analysis is concerned, zonal factors are found to have less impact at high confinements. In Quasi static analysis, the increasing trend of axial thrust with zonal factors could be found even at higher confinements. This could be attributed to the fact that whenever higher lateral translations are induced more forces will be imparted on the liner. But it is clear that there is not much change in absolute values after a particular depth (in this case 16.5m). This mentioned trend for axial thrust in pseudo static and quasi static analysis are shown in "*Fig.14*&15". In these analysis also it could be found that quasi analysis gives conservative values for axial forces (like bolt forces) in comparison with pseudo static analysis. The maximum values for axial thrust experienced in quasi static analysis are as high as 4 times of the respective values obtained in pseudo static analyses. The location of maximum thrust values induced during quasi static and pseudo static analyses around the excavation is shown in "*Fig.16*". Lateral translation effect on liner thrust during quasi analysis is apparently evident.

As far as moments are concerned their absolute values are very low (as low as 17kNm/m). However their values are more in Quasi Static analysis than the corresponding values obtained in Pseudo static analysis.

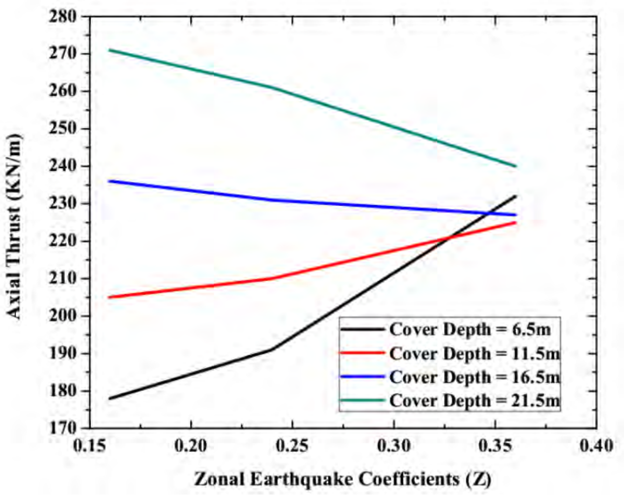

Fig.14. Variation of Axial Thrust with Zonal Factors at Different Cover Depths –Pseudo Static Analysis

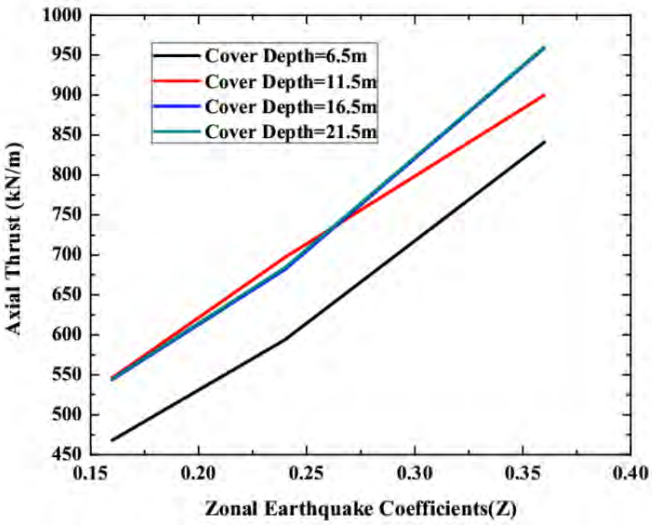

Fig.15. Variation of Axial Thrust with Zonal Factors at Different Cover Depths –Quasi Static Analysis

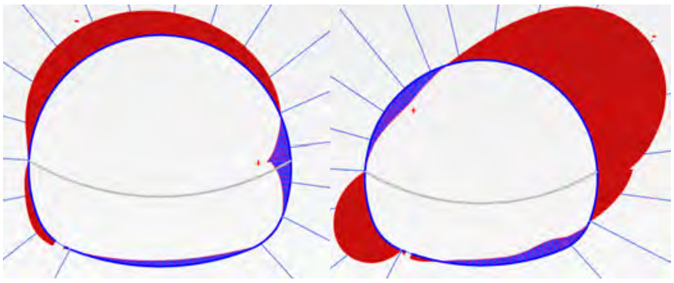

Fig.16. Liner Forces Distribution around Excavation (1) Pseudo Analysis (Left) (2) Quasi Analysis (Right)

C. Liner Capacity Plot

As mentioned above conservative values are obtained for axial thrust and moments during quasi static analysis in comparison with the corresponding pseudo static analysis. The intention of the study is to ascertain whether the primary support could safely handle these seismic structural forces. The maximum thrust force encountered on the tunnel liner (*N*) = 960kNm/m. The maximum moment encountered on the tunnel liner (*M*) =17kNm/m. Both these values are obtained at a cover depth of 21.5m in *Zone V* during quasi static analysis. Now it will be appropriate to ascertain whether the 150mm thick *SFRS* liner could handle this force or not.

For the calculation of capacity of liner element, the hardened shotcrete properties (28days) have to be defined. For the same Modulus (E) =20GPa, compressive strength = 40MPa and tensile strength =4MPa are used. For important tunnel excavation problems, a factor of safety 2 could be adopted [14]. The Moment- Thrust Interaction plot for the shotcrete liner is shown below "*Fig.17*".

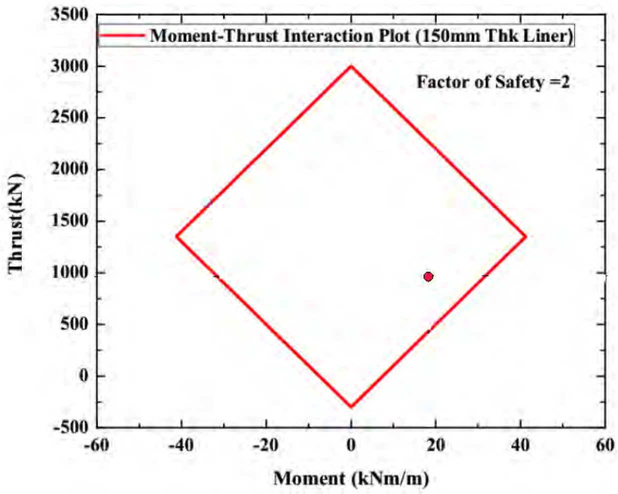

Fig.17. Thrust Interaction Plot for 150mm thick SFRS Liner Element

It could be clearly seen that primary supports, both bolts and primary liner (shotcrete) could safely handle the induced loads even at seismic condition. The only fact to be ensured here is the durability of these primary supports, so that some economization could be achieved for the final concrete liner.

VIII. Conclusions

In static conventional excavation problems, the bolt forces and axial thrust on the primary support tends to increase, courtesy higher stresses due to confinement. In both Quasi static and Pseudo static analysis the bolt forces on primary supports tend to increase during traversal from *Zone III* to *Zone V*. However, this rate of increase tends to diminish at higher cover depths. In Quasi static analyses the axial thrusts tend to increase for a constant cover depth during traversal from *Zone III* to *Zone V*. However, at higher cover depths their absolute values become constant. This tends to agree the fact that influence of seismic effect becomes less pronounced at high cover depths. In Pseudo static analysis the rate of increase of axial thrust decrements as a traversal is done from *Zone III* to *Zone V*. However, at high cover depths it could be seen that rate of increase reverses its trend. This also agrees with the fact that at higher cover depths seismic effects may be ignored. Quasi static analysis tends to give more conservative results in comparison with the pseudo static analysis as far as liner thrust and bolt forces are concerned. Primary support systems (Bolts and *SFRS* elements) could safely handle seismic loads computed by either approaches. Hence if durability of these systems could be ensured in design life some economization could be achieved for the final liner.

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