Numerical modeling and monitoring of blast-induced ground vibration of the Kebasen railway tunnel in Indonesia

Simon Heru Prassetyo
Mining Engineering, Institut Teknologi Bandung, Bandung, Indonesia

Ganda Marihot Simangunsong
Mining Engineering, Institut Teknologi Bandung, Bandung, Indonesia

Yofi Okatrisza
Commitment-Making Officer for Cirebon-Kroya Double-Track Railways, Directorate General of Railways, Ministry of Transportation of the Republic of Indonesia, Jakarta, Indonesia

Imam Subchi
Project Manager for the Kebasen Tunnel Construction, PT. Adhi Karya (Persero) Tbk, Jakarta, Indonesia

ABSTRACT: This paper presents the findings of numerical analysis and monitoring of blast-induced ground vibration of a rock tunnel for the Kebasen railway tunnel in Indonesia. The numerical analysis uses the blast load approach that calculates the magnitude of the equivalent blast load to be imposed on the excavation boundary according to the blasting sequence. This dynamic simulation is conducted using 2-D numerical software called FLAC. The results of the numerical simulation are consistent with the PPV and ground vibration curve from the field monitoring and provide site-specific PPV equations for the Kebasen tunnel. This paper suggests that a safe blasting design can be produced based on preliminary numerical analysis, particularly when site-specific blast-induced ground vibration data are not readily available.

Keywords: Rock tunnel, blast load, blast-induced ground vibration, peak particle velocity, numerical model, Kebasen railway tunnel.

1 INTRODUCTION

The drill-and-blast technique has been a common excavation method for hard rock tunnels and underground caverns in Indonesia. However, blasting generates shock waves and ground vibration that may generate seismic hazard concerns to the existing surrounding structures, such as residential housing, dams, bridges, buildings, or even the existing tunnel. Hence, ensuring the safety of the adjacent structures becomes the main issue that must be solved by the contractor and owner.

The study in this paper was motivated by blast-induced ground vibration of the Kebasen tunnel, a recently completed double-track railway tunnel located in Kebasen district, Central Java Province, Indonesia. The tunnel was built to replace the existing single-track tunnel that was located adjacent to the new Kebasen tunnel. Because the Kebasen tunnel was excavated through 2 hills, it is comprised of 2 tunnels, namely Tunnel 1 and Tunnel 2. The tunnel cross-section is horseshoe-shaped with a dimension of 10.5 m x 9.1 m and is excavated through strong andesite of UCS = 50-140 MPa. Because of the presence of this andesite rock, the advance of the tunnel when using a twin-header excavator was greatly reduced from 1 m/day to only 5 cm/day. Hence, the drill-and-blast technique had to be used. However, before its construction began, there were concerns regarding the effects of blasting on the stability of the existing adjacent tunnel near Tunnel 1 and the existing road cut slope near Tunnel 2 (Figure 1). Numerical modeling was then performed as a preliminary assessment.

This paper presents the results of blasting simulation of the Kebasen tunnel using the equivalent blast load approach. The load is then imposed on the excavation boundary according to the blasting sequence. This dynamic simulation was carried out in a 2-D numerical program called Fast Lagrangian Analysis of Continua or FLAC (Itasca 2020). The resulting peak particle velocity (PPV) from the numerical model is then validated by the result of ground vibration monitoring.

Figure 1. The location of the Kebasen tunnel (Tunnel 1, Tunnel 2, and the existing tunnel).

2 BLAST LOAD AND BLASTING SEQUENCE

The magnitude of the equivalent blast load σ_{eqv} is calculated as

$$\sigma_{eqv}(\text{in MPa}) = \sigma_p \frac{0.03\emptyset_L}{S} \tag{1}$$

where $\emptyset_L$ is the blast hole diameter (45 mm), S is the tunnel perimeter (32 m), and σ_p is the blast pressure (MPa) calculated as

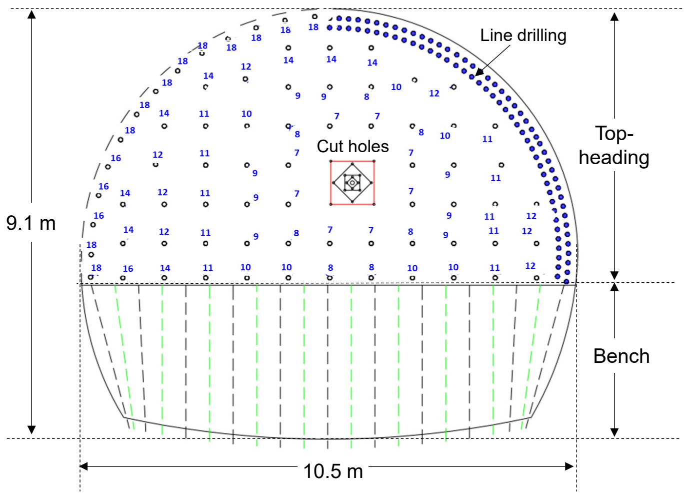
$$\sigma_p(\text{in MPa}) = \frac{\rho \times C_p}{10^6}\frac{PPV}{1000} \tag{2}$$

where ρ is the rockmass density (kg/m^3), C_p is the P-wave velocity of the rockmass (m/s), and PPV is the predicted peak particle velocity at the tunnel boundary (mm/s) calculated as

$$PPV(\text{in mm/s}) = 372.3 \times SD^{-1.17} = 372.3 \times \left(\frac{R}{\sqrt{Q}}\right)^{-1.17} \tag{3}$$

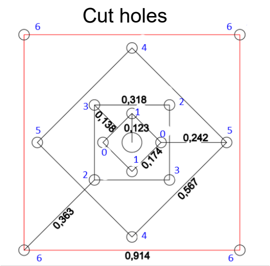
where SD is called the scaled distance with R = 0.5 m and Q is the explosive charge per delay (kg).

Due to safety concerns, the owner and the contractor decided to only blast the top-heading part of the tunnel (Figure 2), while the bench was to be excavated using a rock breaker. The total of 188 holes consisted of 16 cut holes (ø = 45 mm) plus one reamer hole (ø = 82 mm), 87 blast holes (ø = 45 mm), and 85 line drilling holes (ø = 45 mm, uncharged). The length of each cut hole and blast hole was 1.2 m, while that of the line drilling hole was 3 m. Non-electric detonators were used with short-period delays of 100 ms, resulting in a duration of blasting of less than 4 s. Total explosive charge per round was 88.2 kg, yielding a specific charge of 1.2 kg/m^3 (round advance of 1 m or 83%).



Parameter	Roof	Wall	Lifter	Stoping
Burden (m)	0.6	0.9	0.7	0.8
Spacing (m)	0.7	0.7	0.9	0.9
Charge concentration (kg/m)	0.5	1.4	1.4	1.4
Hole depth (m)	1.2	1.2	1.2	1.2
Charge weight/hole (kg)	0.4	1.0	1.0	1.0
Number of holes	87 holes			

Parameter	Line Drilling
Burden (m)	0.2
Spacing (m)	0.2
Charge concentration (kg/m)	-
Hole depth (m)	3
Charge weight/hole (kg)	-
Number of holes	85 holes



Parameter	Cut			
	1	2	3	4
Burden (m)	0.1	0.1	0.2	0.4
Spacing (m)	0.2	0.3	0.6	0.9
Charge concentration (kg/m)	0.8	1.4	1.4	1.4
Hole depth (m)	1.2	1.2	1.2	1.2
Charge weight/hole (kg)	0.6	1.0	1.0	1.0
Number of holes	16 holes			

Figure 2. The blasting sequence of the Kebasen tunnel.

In each delay (sequence), σ_{eqv} reached its value at its increasing time t_R and subsided to zero at its total time t_S (Wang, 1984) according to Eqs. (4) and (5):

$$t_R(\text{in ms}) = \frac{12\sqrt{r^{2-v}}\ Q^{0.05}}{K} \tag{4}$$

$$t_S(\text{in ms}) = \frac{84\sqrt[3]{r^{2-v}}\ Q^{0.2}}{K} \tag{5}$$

where r is the acting radius of the blast load (m), v is the rockmass Poisson's ratio, and K is the rockmass bulk modulus (10^5 Pa). The above equations were adopted from Liang et al. (2013), Liao (1992), and Zhou et al. (2017).

The properties of the andesite rockmass were density $\rho = 2500$ kg/m^3, P-wave velocity $C_p = 5300$ m/s, compressive strength $\sigma_c = 58$ MPa, Young's modulus $E = 7900$ MPa, Poisson's ratio $v = 0.18$, cohesion $c = 3.8$ MPa, friction angle $\phi = 50^o$, and bulk modulus $K = 4100$ MPa. With these properties, based upon the blasting sequence in Figure 2, it was calculated that $\sigma_{eqv} = 0.7$-1.7 MPa with $t_R = 1.7$ ms and $t_S = 2.7$ ms.

3 RESULTS OF NUMERICAL MODELING AND VIBRATION MONITORING

This section presents the results of the blasting simulation of the Kebasen tunnel using the above equivalent blast load imposed at the excavation boundary in the FLAC model.

Figure 3a shows an example of the result of dynamic simulation in FLAC for the scenario as if full-face blasting technique was performed for Tunnel 1. The figure shows that the propagation of blasting vibration from Tunnel 1 to the existing tunnel after 10 ms was around 5-8 mm/s. For this scenario, the maximum ground vibration (PPV) experienced by the existing tunnel was 16 mm/s. While this magnitude is still lower than that of the PPV caused by a passing train (PPV = 18 mm/s), the owner and the contractor decided to not use the blasting technique to excavate Tunnel 1. Moreover, based on this numerical analysis, they also decided to only blast the top-heading part of the tunnel to excavate Tunnel 2 (with the blasting sequence as shown in Figure 2).

Figure 3b shows the propagation of the blasting vibration from Tunnel 2 after 5 ms using the blasting sequence in Figure 2. A numerical monitoring point was placed in the model and located 25 m above the tunnel roof in accordance with the field monitoring point placed during the actual blasting of Tunnel 2. The actual vibration monitoring was carried out using Minimate Plus equipped with a geophone and a microphone. Figure 4 shows the ground vibration curve at this monitoring point. It can be seen that the ground vibration curve resulting from the numerical model in FLAC was in close agreement with that of the actual field monitoring. The PPV from the numerical model was 18 mm/s, while that from the field monitoring was 16 mm/s. This simulation also shows that the permanent displacement experienced by the slope was 3 mm, while that from the actual Total Station monitoring was 2.7 mm. No damage was observed to the road slope and the existing tunnel.

Based on this good agreement between the numerical model and the field monitoring, the blasting of Tunnel 2 was continued and closely monitored. Figure 5 shows the plot of PPV versus SD during the monitoring period. It can be seen from the upper and lower bound regression lines that the result of the numerical model is in line with that of the vibration monitoring. From this figure, the site-specific PPV equations for the blasting of Kebasen tunnel are

$$\text{for mean regression: } PPV(\text{in mm/s}) = 140SD^{-1.1} \tag{6}$$

$$\text{for upper bound regression: } PPV(\text{in mm/s}) = 280SD^{-1.1} \tag{7}$$

$$\text{for lower bound regression: } PPV(\text{in mm/s}) = 70SD^{-1.1} \tag{8}$$

Similar PPV equations have also appeared in Widodo et al. (2022) and Agrawal and Mishra (2019).

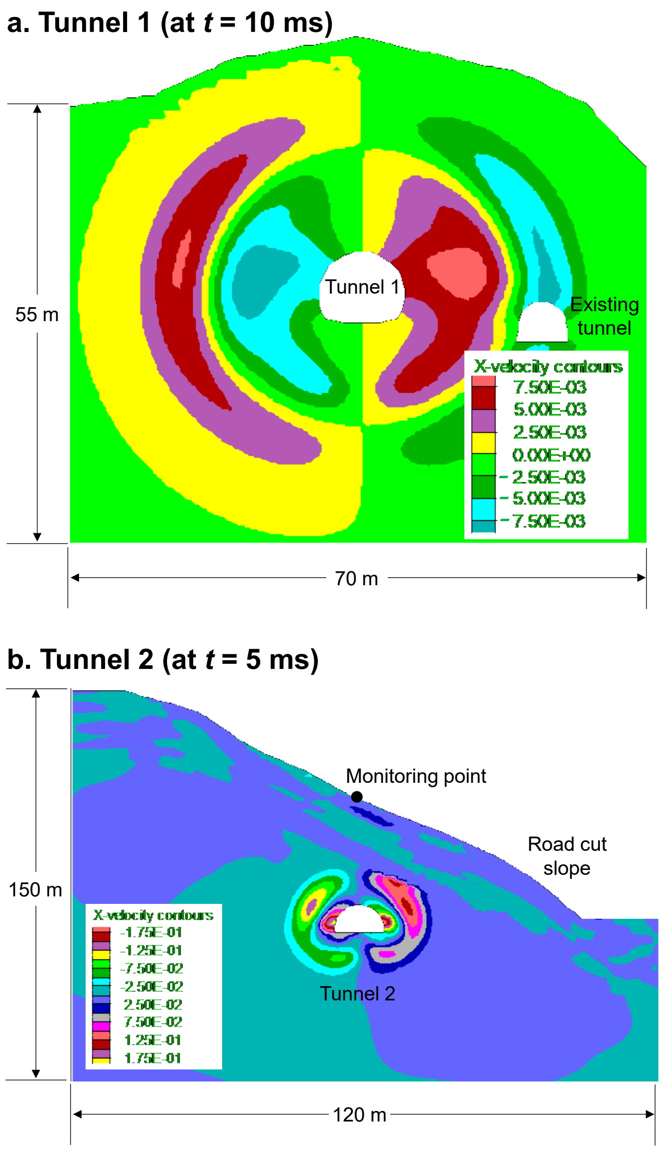


Figure 3. Propagation of ground vibration from the blasting of Tunnel 1 and Tunnel 2 (numerical modeling).

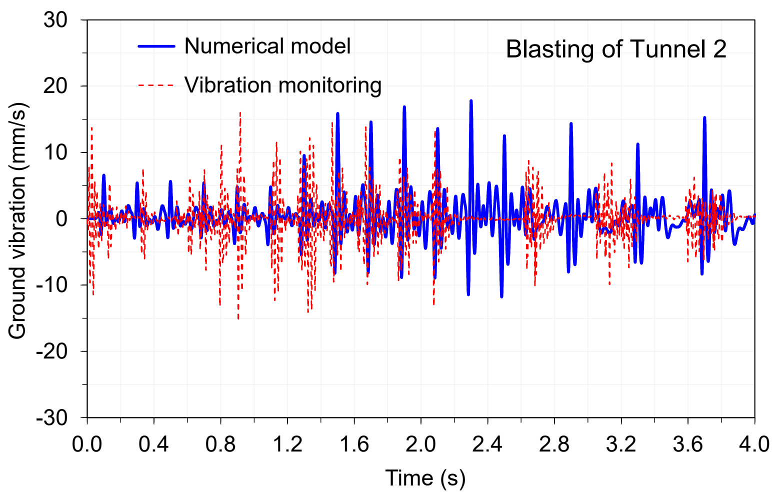


Figure 4. Ground vibration curves from numerical model and actual field monitoring.

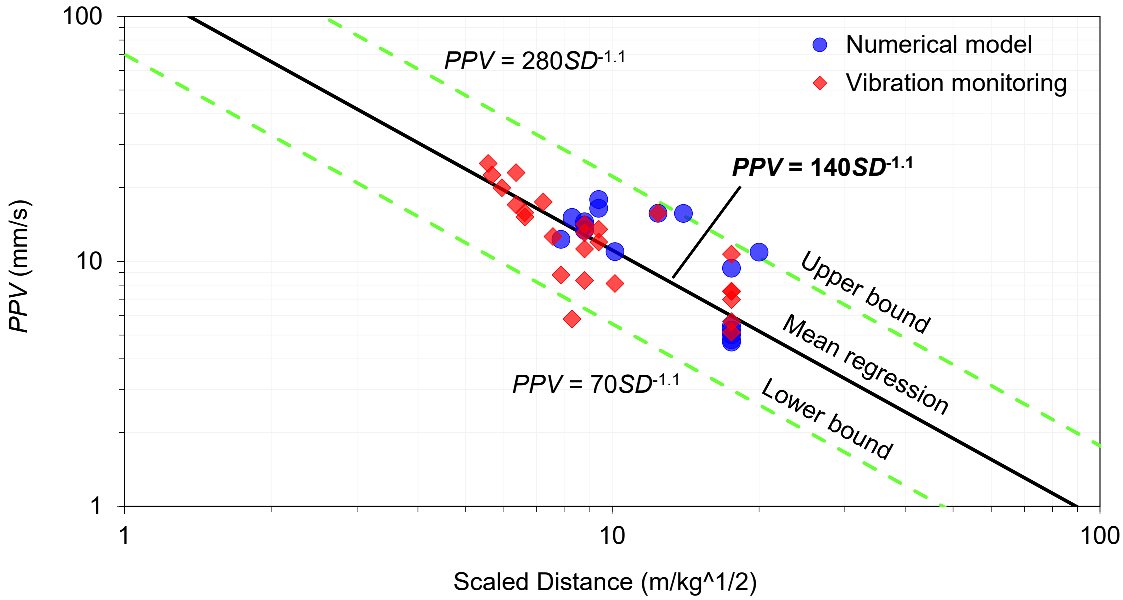


Figure 5. PPV vs. SD of the Kebasen tunnel.

4 CONCLUSION

Using the Kebasen railway tunnel as a case study, this paper shows that the equivalent blast load approach in a numerical model can be applied as a preliminary analysis for tunnel blasting. The results of the numerical simulation are consistent with the PPV and ground vibration curve from the actual vibration monitoring. The results of this paper have been consulted by the owner and the contractor to determine a safe blasting design for the Kebasen tunnel, particularly when site-specific blast-induced ground vibration data (resulting in PPV equations) are not readily available.

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