Jour. of the KGS, Vol. 18, No. 3. June 2002, pp. 51~59

Tunneling-induced Building Damage Risk Assessment System

터널굴착에 따른 인접건물 손상위험도 평가시스템

Park, Yong-Won[*1] 박용원
Yoon, Hyo-Seok[*2] 윤효석

요지

이 논문에서는 터널막장 주변지반의 3차원적 지반거동을 고려한 인접건물의 손상위험도 평가시스템 개발에 관한 내용을 다루었다. 이 시스템은 크게 건물 및 지반정보 모듈, 계측데이터 모듈, 침하평가모듈 및 건물 손상평가모듈로 구성되어 있다. 지반 침하평가 및 건물 손상평가 모듈은 이 시스템의 핵심 모듈로서 Attewell 등(1982)이 제안한 침하평가 모형을 토대로 터널시공으로 인한 침하량 및 범위를 정량적으로 평가한 후, 터널노선에 인접한 건물의 손상위험도를 Mair 등(1996)이 제시한 건물손상 평가방법을 근거로 평가한다. 터널굴착으로 인한 지반거동 평가에서 가장 큰 영향인자인 지반손실률(V_s) 또는 최대침하량($w_{\max}$) 및 변곡점(i)의 위치는 계측자료, 수치해석 결과 그리고 각종 경험식을 사용하여 자동적으로 계산되도록 구축하였다. 한편, 건물 손상평가는 터널막장의 위치를 변화시키며 임의 구간의 인접건물에 대한 손상위험도 평가가 수행될 수 있는 기능을 부여하였다. 개발된 시스템의 검증은 Boscardin과 Cording(1989)이 워싱턴 DC의 매트로 터널에 인접한 2층 조적식 건물의 계측사례를 적용하여 수행하였다.

Abstract

This paper deals with development of a damage risk assessment system for adjacent buildings to under-passing tunnel face considering 3D-ground movement. The system consists of building and ground information module, monitoring data module, settlement evaluation module, and building damage risk assessment module. The major modules, settlement evaluation module and building damage assessment module, are based on settlement estimation model suggested by Attewell et al (1982) and the building damage assessment method by Mair et al. (1996). After estimating 3D-ground movements due to tunneling with settlement evaluation module, damage assessment for buildings is performed using building damage risk assessment module. The developed system has two major functions; 1) calculation of 3D-settlement with ground loss (V_s) or maximum settlement ($w_{\max}$) and inflection point (i) using various empirical formulae, monitoring data, numerical results, and so on; 2) assessment of damage risk for adjacent buildings of arbitrary section with position change of tunnel face. The field data given by Boscadin and Cording (1989) for the case of two-storied masonry building near the Metro tunnel in Washington D.C. was simulated to verify the applicability of the developed system.

Keywords : Building damage, Ground movement, Risk assessment, Settlement, Three demensional, Tunneling

---

[*1] Member, Professor, Dept. of Civil & Environmental Engrg., Myongji Univ., Yongin, Korea (ywpark@mju.ac.kr)
[*2] Member, Ph D. Student, Dept. of Civil & Environmental Engrg., Myongji Univ., Yongin, Korea

1. Introduction

One of the important and difficult problems in tunnel design and construction in urban area may be estimation of the possible damage level of adjacent buildings due to new tunneling. Some researchers (Burland et al. 1977, Boscardin et al. 1989, Mair et al. 1996) performed studies on damage assessment of buildings due to tunneling-induced ground movements. Conventionally, Gaussian normal probability form and normal cumulative probability form have been used to define settlement trough at green-field condition for transverse and longitudinal direction of tunnel line, respectively. And the building damage has been assessed by the relationship between damage categories (angular distortion and deflection ratio) and horizontal strain. However, the method deals with only the walls of buildings at transverse section along the tunnel line after 100%-settlement developed. In fact, ground movement due to tunnel excavation starts before the tunnel face reaches and converges after the tunnel face passes away, which makes adjoining surface buildings to experience 3D-dynamic wave of deformation during tunneling. And the conventional methods of building damage assessment are not effective to evaluate damage of adjacent buildings due to 3D-ground movements near tunnel face. This study aims at solving this problem by a system developed to evaluate damage risk of each wall of adjacent buildings considering 3D-ground movement due to under passing of the tunnel face.

2. Ground Movements and Building Damage Estimation

2.1 Prediction of Ground Movements

Ground movements are estimated in this system based on the procedure adopted by Attewell et. al. (1982). They suggested Gaussian normal probability form for transverse profile (y-axis) and cumulative probability form for longitudinal profile (x-axis) to calculate the vertical and horizontal displacement at any point on a deforming ground due to tunneling, which are shown in Eqs(1)~(4).

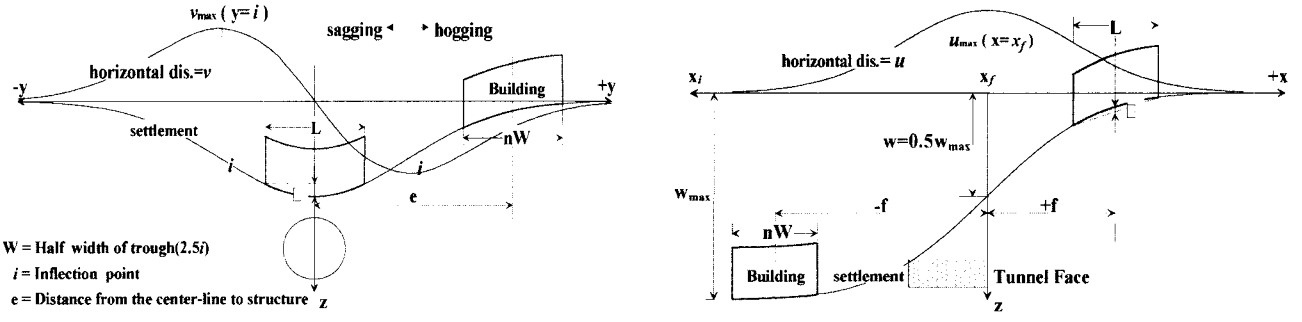
$$w = w_{\max}\exp\left[\frac{-y^2}{2i^2}\right]\left\{G\left(\frac{x-x_i}{i}\right)-G\left(\frac{x-x_f}{i}\right)\right\} \tag{1}$$

$$G(\alpha) = \frac{1}{\sqrt{2\pi}}\int_{-\alpha}^{\alpha}\exp\left[\frac{-\beta^2}{2}\right]d\beta \tag{2}$$

$$v = \frac{y}{z}w \tag{3}$$

$$u = \frac{w_{\max} i}{\sqrt{2\pi} z}\exp\left[\frac{-y^2}{2i^2}\right]\left\{\exp\left[\frac{-(x-x_i)^2}{2i^2}\right]-\exp\left[\frac{-(x-x_f)^2}{2i^2}\right]\right\} \tag{4}$$

where, u, v, w are the ground displacement in the x, y, z directions, respectively. $w_{\max}$ and i are the maximum settlement of the transverse settlement trough and the distance from tunnel center line to inflection point, respectively. x_i and x_f are the distance from the origin to the beginning point of tunnel and the tunnel face, respectively. The value of $G(\alpha)$ can be determined from the standard probability table. Fig. 1 shows the ground settlement profiles of transverse section to tunnel line and longitudinal section along tunnel line, the relationship



(a) Settlement profile of transverse section

(b) Settlement profile of longitudinal direction

Fig. 1. Settlement profiles and building deformation

Table 1. Representative formulae for inflection point

No.	Researcher	Equation	Reference
1	Peck (1969)	$i=0.2(D+z)$	
2	O'Reilly et al (1982)	$i=0.43z+1.1m$ / $i=0.28z-0.1m$	Clay / Sand
3	Atkinson et al (1977)	$\frac{i}{R}=C_1(z/D)C_2$	$C_1=0.750$ / $C_2=0.125$
4	Clough et al (1981)	$\frac{i}{R}=\left(\frac{z}{D}\right)^n$	$n=0.8$

between settlement and horizontal movement, and types of building deformation for the case of single tunnel.

As shown in Fig. 1, damage assessment for buildings is performed separately at either side of a point of inflection, hogging and sagging, in transverse direction, and at either side of a tunnel face in longitudinal direction along tunnel line. Maximum settlement $w_{\max}$ and inflection point i are necessarily to be determined to assess damage level of settlement for buildings against tunneling-induced 3D-ground movement. Studies by researchers such as O'Reily and New (1982), Clough and Schmidt (1981) manifested that the influence factors for inflection point are ground condition, tunnel depth, method of tunnel excavation. Table 1 shows some suggestions for i.

2.2 Estimation of Building Damage

The assessment method for building damage proposed by Mair et al (1996) was adopted in this system, in which a building is treated as an idealized beam with span L and height H deforming under a central point load to give a maximum deflection △. As shown in Fig. 2, the deflection ratios $\triangle/L$ for beam to the bending strain (ε_b) and diagonal strain (ε_d) are calculated using Eqs. (5) and (6), respectively.

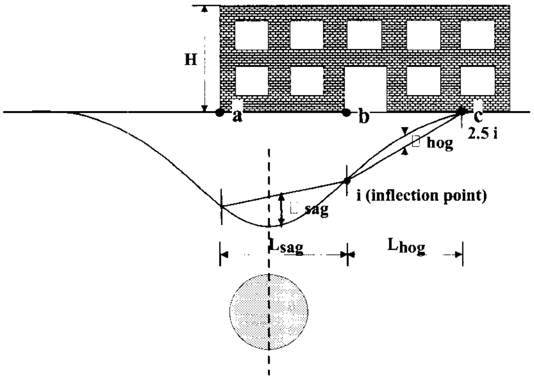


Fig. 2. Definition of deflection ratio

$$\frac{\triangle}{L}=\left\{\frac{L}{12t}+\frac{3IE}{2tLHG}\right\}\varepsilon_b \tag{5}$$

$$\frac{\triangle}{L}=\left\{1+\frac{HL^2G}{18IE}\right\}\varepsilon_d \tag{6}$$

where, E and G are respectively the Young's modulus and shear modulus of the building assumed to be acting as a beam. I is the second moment of area of the equivalent beam (i.e. $H^3/12$ in the sagging zone and $H^3/3$ in the hogging zone), and t is the furthest distance from the neutral axis to the edge of the beam (i.e. H/2 in the sagging zone and H in the hogging zone).

The horizontal ground strain (ε_h) due to tunneling is calculated as the average horizontal strain across a section of building. The average horizontal strain is combined with either the bending or diagonal strain obtained from Eqs. (5) and (6), and the assessment of building damage is performed using Eqs. (7) and (8). Usually the maximum combined tensile strain will occur in the hogging zone because the horizontal ground strain is tensile.

$$\varepsilon_{bt}=\varepsilon_h+\varepsilon_b \tag{7}$$

$$\varepsilon_{dt}=0.35\varepsilon_h+\left[(0.65\varepsilon_h)^2+\varepsilon_d^2\right]^{0.5} \tag{8}$$

A simplified damage assessment chart, directly in terms of $\triangle/L$ and ε_h, presented by Burland (1995) is shown in Fig. 3 for the case of $L/H=1$. This chart can be used for the hogging mode. And the classification of damage for masonry or brickwork structures is given in Table 2, which was first put forward by Burland et al (1977). Boscardin and Cording (1989) showed that the damage categories in Table 2 are related to the magnitude of tensile strain induced in the building, as shown in Table 3.

Table 2. Classification of damage for masonry or brickwork (Burland et al, 1977)

Category of damage	Normal degree of severity	Description of typical damage
0	Negligible	Hairline cracks less than about 0.1mm.
1	Very Slight	Fine cracks which are easily treated during normal decoration. Damage generally restricted to internal wall finishes. Close inspection may reveal some cracks in external brickwork or masonry. Typical crack widths up to 1mm.
2	Slight	Cracks easily filled. Re-decoration probably required. Recurrent cracks can be masked by suitable linings. Cracks may be visible externally and some repointing may be required to ensure weathertightness. Doors and windows may stick slightly. Typical crack widths up to 5mm.
3	Moderate	The cracks require some opening up and can be patched by a mason. Repointing of external brickwork and possibly a small amount of brick-work to be replaced. Doors and windows sticking. Service pipes may fracture. Weathertightness often impaired. Typical crack widths are 5 to 15mm or several greater than 3mm.
4	Severe	Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows. Windows and door frames distorted, floor sloping noticeably. Walls leaning or bulging noticeably, some loss of bearing in beams. Service pipes disrupted. Typical crack widths are 15 to 25mm but also depends on the number of cracks.
5	Very Severe	This requires a major repair job involving partial or complete rebuilding. Beams lose bearing, walls lean badly and require shoring. Windows broken with distortion. Danger of instability. Typical crack widths are greater than 25mm but depends on the number of cracks.

Table 3. Relationship between category of damage and limiting tensile strain (after Boscardin and Cording, 1989)

Category of damage	Normal degree of severity	Limiting tensile strain $\varepsilon_{\lim}$(%)
0	Negligible	0-0.05
1	Very Slight	0.05 - 0.075
2	Slight	0.075 - 0.15
3	Moderate	0.15 - 0.3
4 to 5	Severe to Very Severe	〉0.3

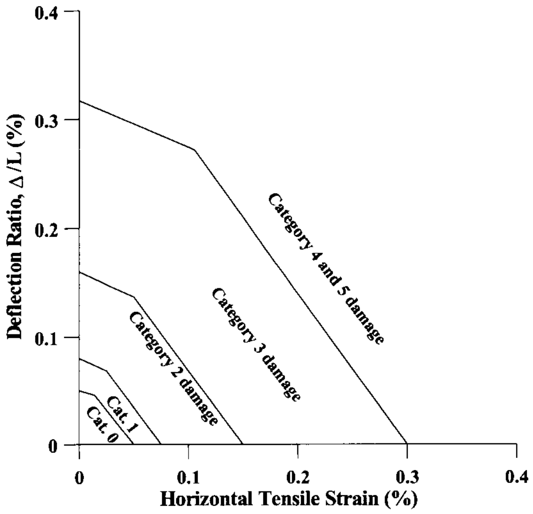


Fig. 3. Damage category chart for L/H=1, hogging mode (Burland, 1995)

3. Tunneling-induced Building Damage Risk Assessment System

3.1 General

The system was developed using Visual Basic to maximize function of GUI (Graphic User Interface) and OpenGL graphic library for processing graphic function that is operated in Windows98. As shown in Fig. 4, the system consists of building information input module, monitoring data input module, settlement evaluation module, building damage assessment module.

The descriptions of each module involved in the application of the developed system are as follows.

(1) In building information module, information of tunnel

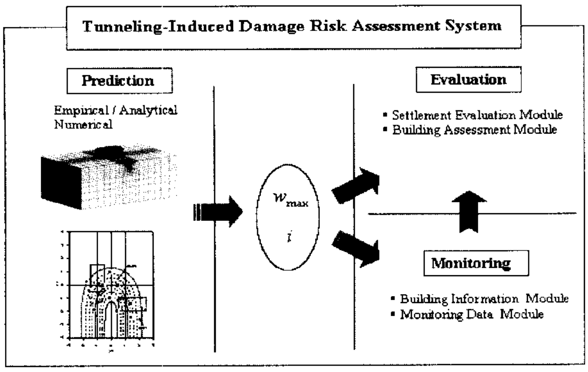


Fig. 4. Organization of system

and buildings, ground condition, tunnel alignment, and so on are stored as input data to this system. This module can be severed as a database for tunnel design. Examples of building information along the tunnel line are shown in Fig. 5(a) and (b).

(2) Settlement evaluation module computes the magnitude and extent of 3D-ground movements using the settlement estimation model suggested by Attewell et al (1982). This module computes 3D-settlement with ground loss (V_s) or maximum settlement ($w_{\max}$) and inflection point (i) using various empirical and semi-empirical formulae, monitoring data, numerical results, and so on. And surface settlement data from monitoring can be fed back into this module, from which the maximum settlement and inflection point are recalculated and used in a subsequent evaluation. The results of settlement evaluation can be visualized in the form of a contour, transverse settlement profile and longitudinal settlement profile as shown in Fig. 5(a)~(d).

(3) Building damage assessment module calculates the maximum tensile strain of a selected building, the bending, diagonal and horizontal strain of each building wall by applying the predicted ground movements at the location of tunnel face. And then the calculated maximum tensile strain is referred to the relationship between damage category and limiting tensile strain in Tables 2 and 3. Figs. 5(e) and (f) show the result of damage assessment and deformation of No.1 building in preliminary step, respectively.

(4) In monitoring data module, the monitored surface settlement data from monitoring are stored and analyzed, and the maximum settlement and inflection point are revaluated and then used in settlement evaluation module. Examples of monitoring data module are shown in Figs. 5(g) and (h).

3.2 Characteristics of System

In this system, the damage estimation using various factors such as angular distortion, deflection ratio, relative settlement, and horizontal displacement can be performed for each wall of buildings that are adjacent to tunneling line. Also, bending and diagonal strain that might occur in buildings can be calculated as well as risk of building damage due to tunneling-induced settlement by the developed system.

The characteristics of settlement risk management system for adjacent buildings against tunneling are as follows;

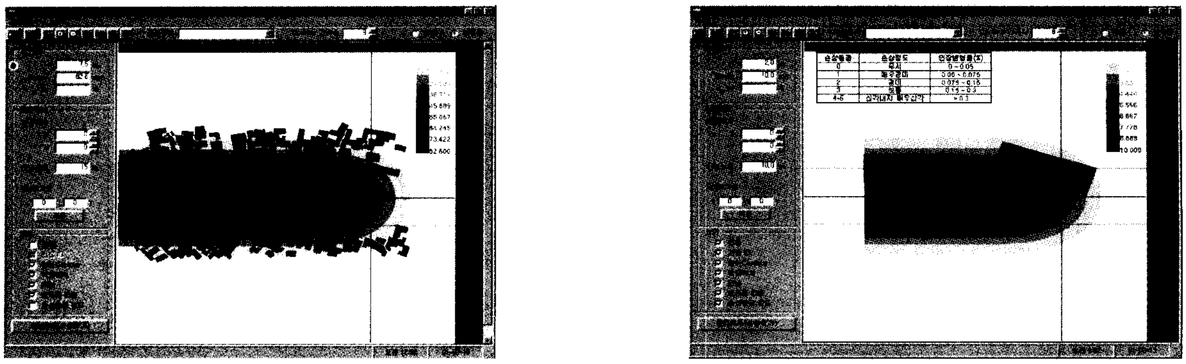
- Offer building information around tunnel route (zoom in/out)
- Display settlement contour at tunnel face
- Display transverse and longitudinal settlement profile to tunnel line
- Analyze monitoring data

4. Case Study for System Application

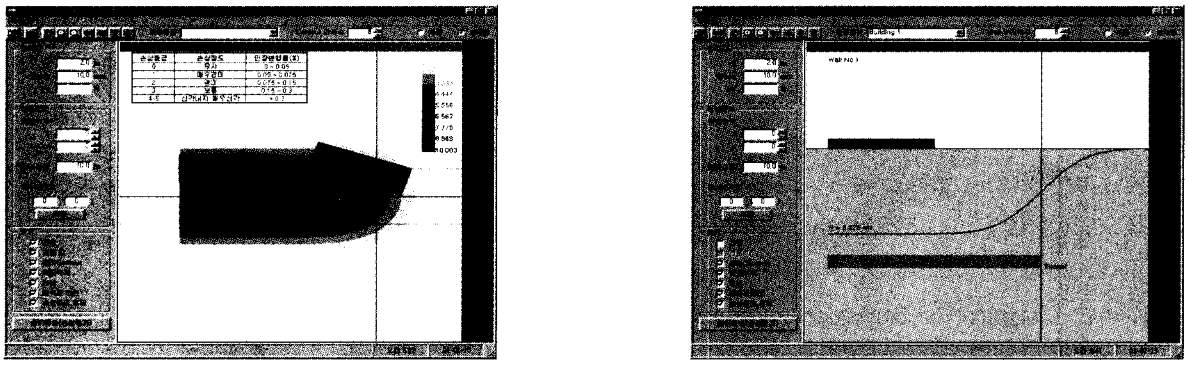
4.1 Field Case Study

As a field case for the examination of the system Boscardin & Cording (1989)'s report which investigated settlement damage of two-storied masonry building adjacent to the Metro subway tunneling in Washington D.C. is applied. As shown in Figure 6(a), twin tunnel with 12.8 meters of pillar width between tunnel centers was constructed by shield method. Tunnel depth was 13.6 meters and distance from inside tunnel to BLD-1 was about 1.5 meters. Figure 6(b) shows relative position of tunneling direction and adjacent buildings. Intersection angle between the buildings and tunnel line was 22°.

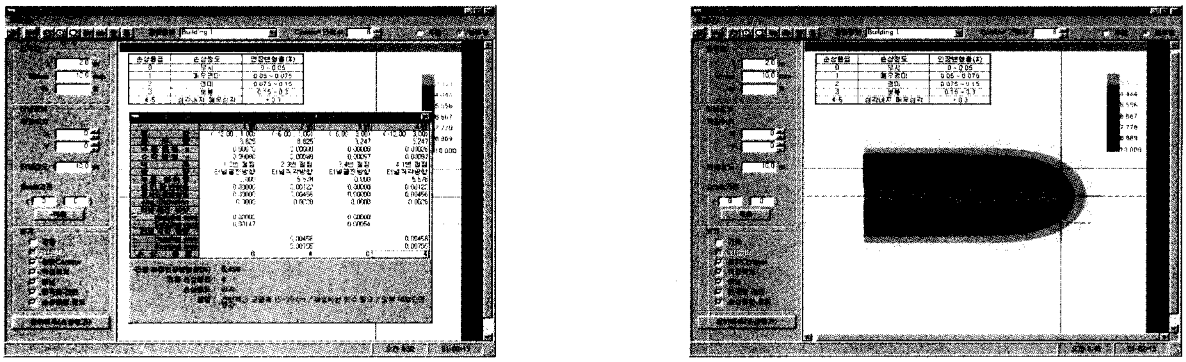
BLD-1 which located near the center of the final



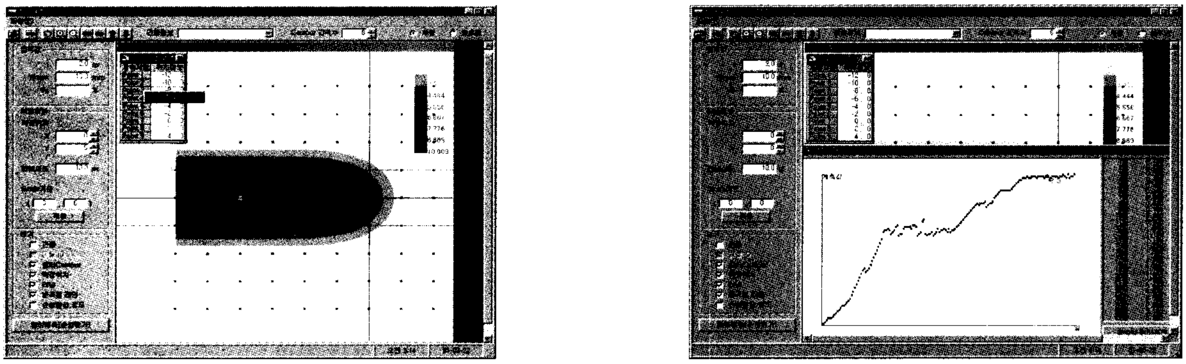
(a) Building information along tunnel line (b) Result of settlement estimation



(c) Transverse settlement trough (d) Longitudinal settlement trough

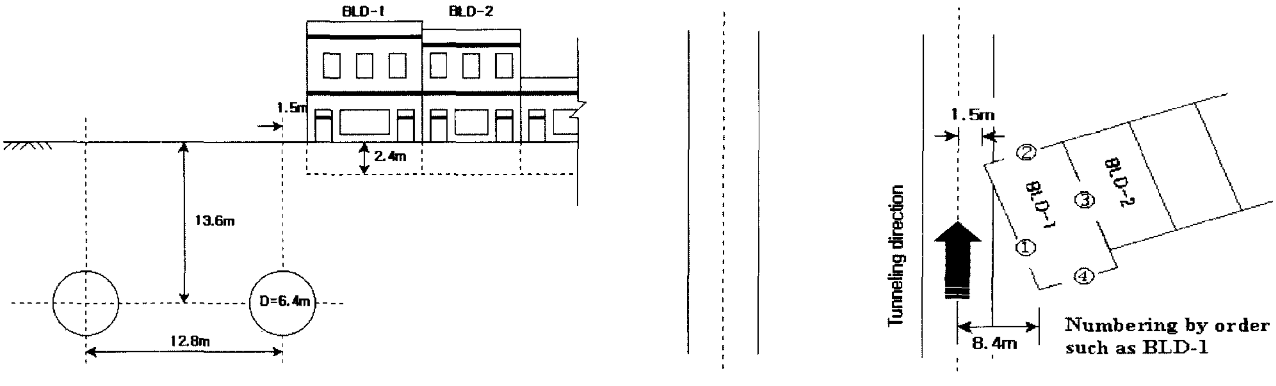


(e) Result of damage assessment (f) Deformation of No.1 building



(g) Monitoring positions on surface (h) Monitoring data

Fig. 5. Application examples of settlement risk management system



(a) Cross section (b) Plane view

Fig. 6. Field case for system application examination

Table 4. Summary of building damage risk (Boscardin & Cording, 1989)

Wall No.	BLD-1	BLD-2
①	-The crack widths increase near at window frame.	-The existent crack widths were increased to about 1.6 mm.
②	-Crack widths of about 0.8 mm happened in neighborhood of entrance. -Crack width of about 3 mm observed the floor and ceiling.	-The existent vertical crack widths were increased to about 1.6 mm.
③	-	-
④	-The existent crack width were increased to 0.4 mm along window frame.	-
Results of damage assessment	-Limiting angular distortion that is proposed by O'Rourke (1976) : 1/1,000 -Monitoring results; Angular distortion of BLD-1: 1/410 Angular distortion of BLD-2: 1/2,000 -Assessment results; BLD-1: 4 grade (Severe) BLD-2: 1 grade (Very Slight)	

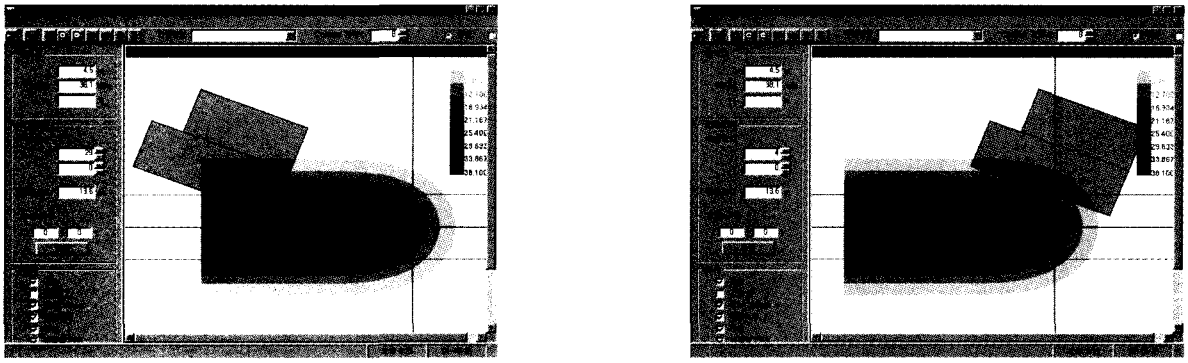
settlement trough was at compression zone (sagging) and showed very little horizontal tensile strain. BLD-2 which located near the edge of the surface settlement trough was at the zone of horizontal tensile strain (hogging). The horizontal tensile stain in BLD-2 was concentrated in a single vertical crack between No.1 and 2 walls. The existent hairline crack opening was 3.0 mm in the first floor and 5.0 mm in the second floor. Tilting of BLD-1 caused 10 mm separation between two buildings at the top of the bearing walls. The shearing distortions in BLD-1 enlarged the opening of cracks by 0.8 mm in the mortar joints between bricks above the entrance and windows on No.2 wall. Table 4 shows summary of damage risk of buildings.

4.2 Application of the Developed System

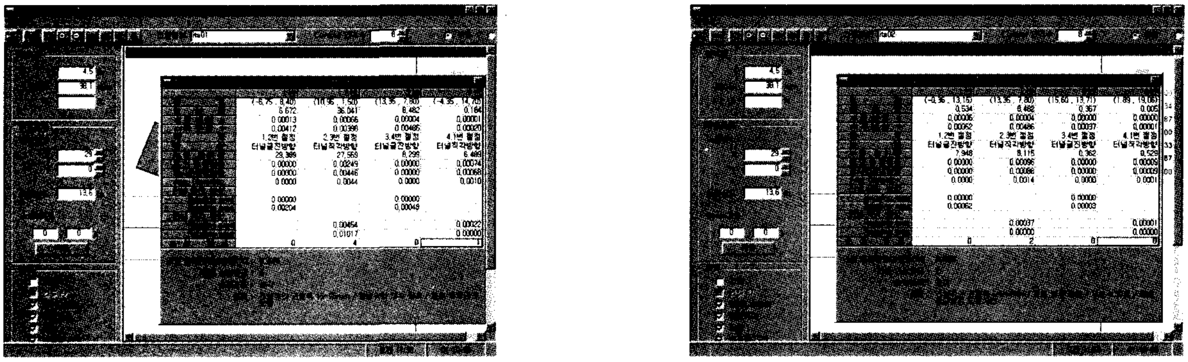
The results of analysis of the aforementioned case by the developed system are compared with the field observation data. In the analysis, the maximum settlement and the position of inflection point were decided as 38.1 mm and 4.5 m, respectively from field observation results. The analysis results are summarized in Table 5. The damage grade of BLD-1 is grade-4 at No.2 wall, and is grade-2 for BLD-2 at No. 2 wall when tunnel face passed the building completely (case-1). These results are somewhat conservative comparing with the results of field observation. This may be why the analysis was carried out for green-field condition without considering the relative rigidity of ground and building and boundary conditions. Nevertheless, from the results, it can be thought that the proposed system can be an effective tool for pre-assessment of building damage due to tunneling at the stage of plan and design of tunnel with less cost and time. At the next stage, the system should be extended for such factors mentioned above to be considered.

Table 5. Summary of analysis result

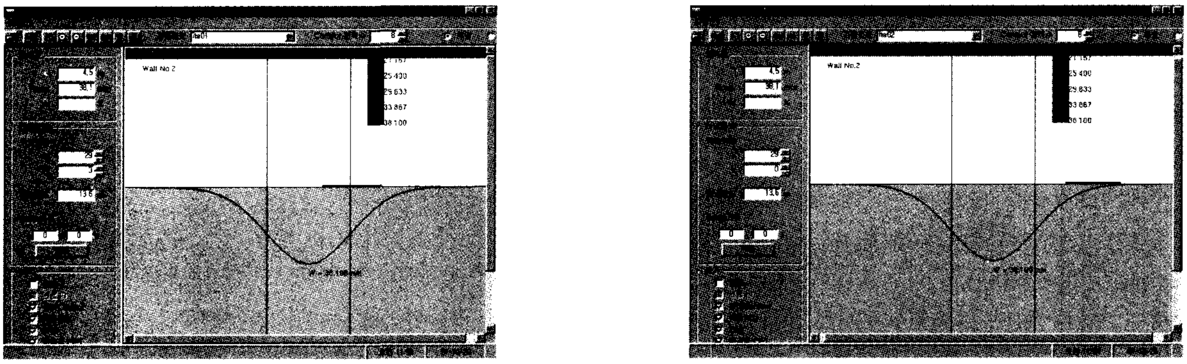
Classification	BLD-1	BLD-2
Case-1	- Max. damage occurrence in No.2 wall - Angular distortion; 1/230 - Deflection ratio; 0.454% - Total tensile strain; 0.45% - Damage Category; 4 grade	- Max. damage occurrence in No.2 wall - Angular distortion; 1/715 - Deflection ratio; 0.037% - Total tensile strain; 0.096% - Damage Category; 2 grade
Case-2	- Max. damage occurrence in No.1 wall - Angular distortion; 1/1,000 - Deflection ratio; 0.022% - Total tensile strain; 0.073% - Damage Category; 1 grade	- No damaged wall



(a) Case-1 (b) Case-2



(c) Damage assessment results of BLD-1 (Case-1) (d) Damage assessment results of BLD-2 (Case-1)



(e) Transverse settlement profile of BLD-1 (f) Transverse settlement profile of BLD-2

Fig. 7. Application of proposed system to field case

5. Conclusion

This study was performed to develop a risk manage system for building damage induced by new tunneling in urban areas. The results of study are summarized as follows;

(1) The proposed system can be used to estimate 3D-ground movement near tunnel face under green-field condition.

(2) Tunneling-induced building damage risk can be assessed in 3D-condition.

(3) Comparing the result of analysis with the proposed system and field observation data, the proposed system shows somewhat conservative results.

(4) At the next stage of system development, factors such as ground conditions, excavation method, relative rigidity of ground and building, and method of determination of inflection point are to be considered in the system.

References

1. Atkinson, J. H. and Potts, D. M. (1977), "Subsidence above shallow tunnels in soft ground", *Journal of Geotechnical Eng. Div.* Proc. ASCE, Vol.103, No.GT4, pp.307-325.
2. Attewell, P. B., Yeates, J. and Selby, A. R. (1986), *Soil Movements Induced by Tunnelling and Their Effects on Pipelines and Structures,* Blacki and Son Ltd., London.
3. Boscardin, M. D. and Cording, E. J. (1989), "Building Response to Excavation-Induced Settlement", *Journal of Geotechnical Engineering,* ASCE, Vol.115, No.1, pp.1-21.
4. Burland, J. B. (1995), "Assessment of risk of damage to buildings due to tunneling and excavations", Invited Special Lecture to IS-Tokyo '95: 1st Int. Conf. On Earthquake Geotechnical Engineering.
5. Burd, H. J., Houlsby, G. T., Augarde, C. E., and Liu, G. (2000), "Modelling tunnelling-induced settlement of masonry buildings", *Proceedings of the Institution of Civil Engineers,* Geotechnical Engineering, 143, pp.17-29.
6. Burland, J. B. and Wroth, C. P. (1974), "Settlement of buildings and associated damage", *Proceedings of a Conference on Settlement of Structures,* Cambrige, pp.611-654.
7. Burland, J. B., Broms, B., and De Mello, V.F.B. (1977), "Behaviour of foundations and structures - SOA Report", Session 2, *Proc. 9th Int. Conf. SMFE,* Tokyo, pp.495-546.
8. Clough, G. W. and Schmidt, B. (1981), "Design and performance of excavations and tunnels in soft clay", In *Soft Clay Engineering,* Elsevier, pp.569-634.
9. Kaalberg, F. J. and Netzel, H. (1999), "Amsterdam's North/South Metroline", *Tunnelling and Underground Space Technology,* Vol. 14, No.2, pp.191-210.
10. Mair, R. J., Tayor, R. N., and Burland, J. B. (1996), "Prediction of ground movements and assessment of risk of building damage due to bored tunneling", *Geotechnical Aspects of Underground Construction in Soft Ground,* Balkema, Rotterdam, pp.713-718.
11. O'Reilly, M. P. and New, B. M. (1982), "Settlements above tunnels in the United Kingdom - their magnitude and prediction", *Tunnelling '82,* London, IMM, pp.173-181.
12. Peck, R. B. (1969), "Deep excavations and tunnelling in soft ground", *SOA Report,* 7th Int. Conf. SMFE, Mexico City, State of the Art Volume pp.225-290.

(received on Apr. 16, 2002, accepted on May 28, 2002)