Structural Evaluation of Masonry Sewer Pipes Under Transverse Settlement Due to Tunneling

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Abstract—The intention of this research is to perform structural evaluation on masonry sanitary sewer pipes impacted by tunnel excavation. Until about 1900, it was common to use several layers of brick to construct municipal sanitary sewer lines, and currently thousands of miles of brick sewer lines are still in service. The construction of new underground facilities inevitably disturbs the soil above and will have an impact on the existing masonry sanitary sewer pipes within the zone of influence of the excavation. Masonry pipes are typically susceptible to the longitudinal interaction failure mechanism at the hogging zone of the settlement trough due to their low tensile capacity. In this research standard egg-shaped section for brick sewer were subjected to ground settlements obtained using Attewell method. The greenfield condition is assumed to be applicable with homogeneous ground conditions. The impact of the depth to the tunnel axis and trough width parameter on the settlement trough and associated strains developed in the pipes are subjected to investigation. Tensile strain developed due to axial strain and bending moment are calculated for five different section sizes. Mitigation measures and recommendations are provided

Index Terms—Masonry Sanitary Sewer, Longitudinal Interaction Failure Mechanism, Transverse Settlement Due to Tunneling, Empirical Ground Settlement.

I. INTRODUCTION

Until about 1900, Large sewers were generally constructed of brick. It was common to build the sewer structures as a combined brick masonry and concrete [1],[2]; the concrete was used for the foundation of brick sewer or for reinforcing or backing up brick arches such that inverts were built out of concrete and arches were built out of bricks [3]. Several layers of brick were used to construct municipal sanitary sewer lines. Currently, thousands of miles of brick sewer lines are still in service in cities such as Washington DC, London, Paris, etc [3].

Further development of cities results in the need for improved infrastructure systems. Subsurface structures such as tunnels are becoming more desirable for infrastructure projects as spaces on the ground are limited[4]. The construction of new underground facilities inevitably disturbs the soil above and will have an impact on the existing structures within the zone of influence of the excavation. The response of the existing structures is dependent to their geometry, construction type and overall structural condition [2], [5], [6]. It should be mentioned that typically there is little knowledge among property owners regarding the history of deformations experienced by the

structure.

The relationship between surface settlements and tunnel depth is neither simple nor linear [7]. Typically, ground movements depend on tunnel geometry and depth, excavation methods, workmanship quality, and geotechnical conditions. It is widely accepted that a transverse section of the Greenfield settlement trough can be obtained with acceptable precision by a reversed Gaussian curve.

II. BACKGROUND AND LITERATURE REVIEW

The structural behavior of a buried pipeline impacted by tunneling depends on the interaction of the displaced soil with the lining of the pipeline. Vorster proposes five soil-structure interaction mechanisms for pipeline failure[8]. These failure mechanisms are: (Mechanism 1): Global Greenfield soil settlement; (Mechanism 2): Gap formation below the pipe; (Mechanism 3): Positive downdrag failure - the pipe settles more than the soil; (Mechanism 4): Negative downdrag failure - the soil settles more than the pipe; (Mechanism 5): Longitudinal interaction. Among the aforementioned failure mechanisms masonry pipes are the most susceptible to the longitudinal interaction due to their low tensile capacity.

Longitudinal interaction is referred to the forces transmitted to the pipe as a result of horizontal soil strains. [2] The worst location for pipes is typically within the hogging zones where horizontal strains applies additional tensile strain on the pipe which encourages tensile fracture or joint pullout [8], [3].

The volume loss (V_L) is typically expressed as a percentage of the excavated ground volume. It is assumed that the area bounded by the undisturbed ground surface and settlement trough is equal to the ground volume loss caused by tunneling operations. The volume loss depends on the tunneling technique and excavation practices (such as the speed of the TBM, applied face pressure, and contact grouting timing and pressure).

For masonry structures TMS 602-13 (Building Code Requirements and Specification for Masonry Structures) specifies that the modulus of rupture is approximately 133 psi [9]. A strength factor (ϕ) for non-reinforced masonry is considered as 0.6 for flexural and axial load. This results to approximate allowable tensile stress value of 80 psi. According to ASCE 7-16 (Minimum Design Loads For Buildings and Other Structures), load factor for self-straining loads including movement due to differential settlement shall not be less than 1.0[10]. Therefore, the stresses obtained from the analysis shall be compared to the allowable stress indicated above with consideration of utility age and condition.

Published on April 29, 2019.

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III. METHOD

This research intends to evaluate the standard egg-shaped masonry sewer underground settlement induced by tunnel excavation. The maximum tensile stress induced by axial strain or bending is compared to the allowable limit proposed by TMS 602-13.

A. Evaluation of Ground Settlement

The Attewell method was used to estimate ground surface settlement due to tunneling [11]. The Attewell method is based on an inverted Gaussian distribution method and. This method was initially developed by Peck and has been further investigated by many researchers including Cording, Attewell, and O'Reilly and New [12], [13],[11], and [14].

The Gaussian distribution method proposed by Peck is presented in Eq.1.

$$S(x) = S_{max} \cdot e^{\left(\frac{-x^2}{2i^2}\right)} \quad (1)$$

$$S_{max} = 0.313 \frac{V_L D^2}{i}$$

Where, S is the vertical surface settlement at a horizontal distance x, and S_{max} is the maximum vertical surface settlement, x is the horizontal distance between tunnel center line and the point where settlement is being evaluated, and i is the horizontal distance between the tunnel centerline and point of inflexion along the settlement trough.

i is a function of the depth and is broadly independent of the tunnel construction method. Eq.2 presents the relationship between i and Z_0 as suggested by Attewell (1982).

$$i = K Z_0 \quad (2)$$

Where, Z_0 is the depth to the tunnel axis; and K is the trough width parameter which depends on soil type and behavior. Table I presents the trough width parameter K for different soil types.

TABLE I: TROUGH WIDTH PARAMETER K

Parameter K	
Stiff Clay	0.4-0.5
Soft Silty Clay	0.6-0.7
Sand above GW	0.2-0.3
Sand below GW	0.4-0.5

Horizontal displacement U is related to the settlement S by the expression presented in Eq.3.

$$U(x) = \frac{s(x) \cdot x}{Z_0} \quad (3)$$

The horizontal strain in the transverse direction is obtained by differentiating the horizontal displacement with respect to y and is presented in Eq.4.

$$\varepsilon_h(x) = \frac{dS_h}{dx} = \left(-\frac{x^2}{i^2} + 1\right) \frac{s(x)}{Z_0} \quad (4)$$

Figure 1 presents the schematics of the vertical settlement, horizontal displacement, and horizontal strain. In this figure point of inflexion is also identified which can be used to separate the tension and compression zones.

Fig. 1. Vertical Settlement, Horizontal Displacement, and Strain

Masonry pipes are typically susceptible to the longitudinal interaction failure mechanism at the hogging zone of the settlement trough due to their low tensile capacity.

B. Volume Loss (V_L)

The choice of the volume loss parameter V_L is of considerable importance and the appropriate value is related to experience of the tunneling technique and ground conditions at the particular project for which predictions are required. In this respect, good case history data is vital.

In recent years TBM performance has improved considerably and has reduced volume losses. Earth Pressure Balance Machines (EPBM) has significantly reduced ground movements and volume loss. This research is not intended to study the impact of different volume losses.

C. Parameters of the study

The depth to the tunnel axis (Z_0), trough width parameter (K), will have the most significant impact on the settlement trough. Volume loss and tunnel diameter are assumed as constants. The variables for this study and the steps for variation are presented in Table 2.

TABLE II: PARAMETERS OF THE STUDY

Parameter K	Range	Step
Depth to the tunnel axis (Z_0)	55ft.-125ft.	10ft.
Trough width parameter (K)	0.3-0.8	0.1
Volume loss (V_L)	0.5%	N/A
Tunnel Diameter	25 ft.	N/A

The cover over diameter ratio (C/D) is varying between 1.7to 4.5 in this study.

D. Limitation of Analysis

Attewell method provides results with good agreement with ground recorded data under following conditions: (1) Greenfield conditions should be applicable as the presence of structures might disturb the displacement field induced by tunneling. (2) Single tunnel or multiple tunnels without interactions. (3) Homogeneous ground conditions. (4) Short term conditions as this empirical method does not incorporate soil displacements due to consolidation.

E. Standard Egg-Shaped Section

Five different section sizes have been subjected to analysis which their geometric properties are presented in Table III. The cradle for these structures is assumed to be constructed using a low strength concrete with modulus of elasticity of 1800 ksi. The modulus of elasticity of the vitrified brick is assumed to be 700 ksi. Concrete is chosen as the base material and the brick work is converted into equivalent concrete section.

TABLE III: GEOMETRIC PROPERTIES OF THE INVESTIGATED SEWERS

Section ID	Area (total) in ²	Equivalent Area (Brick) in ²	Area (Cradle) in ²	I _{xx} in ⁴	Y _{Bar} in	C _{brick} in
3 ft	1714.3	266.4	1450.65	317000	18.9	39.1
3 ft-9 in	2078.2	326.4	1759.91	513600	21.4	45.5
4 ft-6 in	2459.9	382.0	2086.72	780800	23.9	52.1
5 ft-9 in	3143.0	472.1	2678.17	1419000	27.8	63.1
6 ft-6 in	3595.3	538.4	3075.86	1954000	30.2	69.7

Schematics of the standard egg-shaped sections are presented in Figure 2.

Fig. 2. Standard Egg-Shaped Section

IV. RESULTS

Table IV present the results of the maximum vertical settlement and horizontal displacement assuming the volume loss of 0.5%.

Fig. 3. Settlement Trough (K=0.4, Z₀=105ft., VL=0.5%)

A. Vertical and Horizontal Displacements

Table IV presents the maximum vertical settlement and horizontal displacements for the investigated cases

introduced earlier.

TABLE IV: ATTEWELL METHOD RESULTS FOR VL=0.5%

K		Z ₀ (ft)							
		5	6	7	8	9	10	11	12
0.3	V(in.)	-0.71	-0.60	-0.52	-0.46	-0.41	-0.37	-0.34	-0.31
	H(in.)	0.13	0.11	0.09	0.08	0.07	0.07	0.06	0.06
0.4	V(in.)	-0.53	-0.45	-0.39	-0.35	-0.31	-0.28	-0.26	-0.23
	H(in.)	0.13	0.11	0.09	0.08	0.07	0.07	0.06	0.06
0.5	V(in.)	-0.43	-0.36	-0.31	-0.28	-0.25	-0.22	-0.20	-0.19
	H(in.)	0.13	0.11	0.09	0.08	0.07	0.07	0.06	0.06
0.6	V(in.)	-0.36	-0.30	-0.26	-0.23	-0.21	-0.19	-0.17	-0.16
	H(in.)	0.13	0.11	0.09	0.08	0.07	0.07	0.06	0.06
0.7	V(in.)	-0.30	-0.26	-0.22	-0.20	-0.18	-0.16	-0.15	-0.13
	H(in.)	0.13	0.11	0.09	0.08	0.07	0.07	0.06	0.06
0.8	V(in.)	-0.27	-0.23	-0.20	-0.17	-0.15	-0.14	-0.13	-0.12
	H(in.)	0.13	0.11	0.09	0.08	0.07	0.07	0.06	0.06

B. Tensile Strain

In this research it is assumed that the pipe lines are perpendicular to the tunnel alignment which can be considered as the worst case since the maximum tensile strain is being developed. Table V present the results of the maximum compressive and tensile strain for the case with volume loss of 0.5%.

Fig. 4. Horizontal Strain (K=0.4, Z₀=105ft., VL=0.5%)

TABLE V: MAXIMUM COMPRESSIVE AND TENSILE STRAIN, (μ ε)

K		Z ₀ (ft)							
		5	6	7	8	9	10	11	12
0.3	Com. ¹	-478	-342	-258	-201	-161	-132	-110	-93
	Ten. ¹	1078	772	580	451	361	296	247	209
0.4	Com. ¹	-360	-258	-194	-151	-121	-99	-83	-70
	Ten. ¹	808	579	435	338	271	222	185	157
0.5	Com. ¹	-289	-206	-155	-121	-97	-79	-66	-56
	Ten. ¹	647	463	348	271	217	177	148	125
0.6	Com. ¹	-240	-172	-129	-101	-81	-66	-55	-47
	Ten. ¹	539	386	290	226	181	148	123	104
0.7	Com. ¹	-206	-147	-111	-86	-69	-57	-47	-40
	Ten. ¹	462	331	248	193	155	127	106	89
0.8	Com. ¹	-180	-129	-97	-76	-60	-49	-41	-35
	Ten. ¹	404	289	217	169	135	111	92	78

¹All the strains are presented are in micro strain.

C. Axial Stresses

The axial stress induced in the masonry pipes due to longitudinal strain are independent of the section sizes and is presented in the Table VI. Cases which the axial tension is above 80 psi tensile capacity of the masonry pipes are highlighted by red font. In these cases, even without consideration of the tension induced due to bending the masonry pipes will experience tensions above threshold proposed by TMS 602-13 (Building Code Requirements and Specification for Masonry Structures).

TABLE VI: AXIAL TENSILE STRESSES (PSI)

K		Z ₀ (ft)						
		55	65	75	85	95	105	115
0.3	Ten.	-334	-240	-181	-141	-113	-92	-77
0.4	Ten.	-252	-181	-136	-106	-85	-69	-58
0.5	Ten.	-202	-144	-109	-84	-68	-55	-46
0.6	Ten.	-168	-121	-91	-70	-56	-46	-39
0.7	Ten.	-144	-103	-78	-60	-48	-40	-33
0.8	Ten.	-126	-90	-68	-53	-42	-35	-29

All the stresses are in psi.

A. Bending Stresses

Bending stresses for the five different section sizes introduced in the Table III are calculated. The maximum tension is occurring in the hogging zone of the settlement curve and will induce tension in the brick which is located on the upper side of the section. The bending stress induced in the masonry pipes are presented in the Table VII. Cases which the tension is above 80 psi tensile capacity of the masonry pipes are highlighted by red font.

TABLE VII: BENDING STRESSES (PSI)

		Z ₀ (ft)							
K		55	65	75	85	95	105	115	125
3 ft Section									
0.3	Ten.	-381	-231	-150	-103	-74	-55	-42	-32
0.4	Ten.	-161	-97	-63	-44	-31	-23	-18	-14
0.5	Ten.	-82	-50	-32	-22	-16	-12	-9	-7
0.6	Ten.	-48	-29	-19	-13	-9	-7	-5	-4
0.7	Ten.	-30	-18	-12	-8	-6	-4	-3	-3
0.8	Ten.	-20	-12	-8	-5	-4	-3	-2	-2
3 ft- 9 in Section									
0.3	Ten.	-444	-269	-175	-120	-86	-64	-49	-38
0.4	Ten.	-187	-113	-74	-51	-36	-27	-20	-16
0.5	Ten.	-96	-58	-38	-26	-19	-14	-10	-8
0.6	Ten.	-55	-34	-22	-15	-11	-8	-6	-5
0.7	Ten.	-35	-21	-14	-9	-7	-5	-4	-3
0.8	Ten.	-23	-14	-9	-6	-5	-3	-3	-2
4 ft- 6 in Section									
0.3	Ten.	-508	-308	-200	-138	-99	-73	-56	-43
0.4	Ten.	-214	-130	-85	-58	-42	-31	-23	-18
0.5	Ten.	-110	-67	-43	-30	-21	-16	-12	-9
0.6	Ten.	-64	-38	-25	-17	-12	-9	-7	-5
0.7	Ten.	-40	-24	-16	-11	-8	-6	-4	-3
0.8	Ten.	-27	-16	-11	-7	-5	-4	-3	-2
5 ft- 9 in Section									
0.3	Ten.	-616	-373	-243	-167	-119	-88	-67	-52
0.4	Ten.	-260	-157	-102	-70	-50	-37	-28	-22
0.5	Ten.	-133	-81	-52	-36	-26	-19	-15	-11
0.6	Ten.	-77	-47	-30	-21	-15	-11	-8	-7
0.7	Ten.	-48	-29	-19	-13	-9	-7	-5	-4
0.8	Ten.	-32	-20	-13	-9	-6	-5	-4	-3
6 ft- 6 in Section									
0.3	Ten.	-680	-412	-268	-184	-132	-98	-74	-58
0.4	Ten.	-287	-174	-113	-78	-56	-41	-31	-24
0.5	Ten.	-147	-89	-58	-40	-28	-21	-16	-13
0.6	Ten.	-85	-51	-34	-23	-16	-12	-9	-7
0.7	Ten.	-54	-32	-21	-14	-10	-8	-6	-5
0.8	Ten.	-36	-22	-14	-10	-7	-5	-4	-3

All the stresses are in psi.

V. CONCLUSION

Due to very low tensile capacity of the masonry sewer it is extremely difficult to maintain stresses within the allowable limits. There are only few cases with high Z₀ value where the combined axial and bending stresses are within 80 psi limit. Increasing the tunnel depth significantly reduced the strains and stresses developed in the masonry sewers. Cases where trough width parameter (K) has higher values the settlement trough is wider, and the maximum settlements are lower, as a result of that, lower maximum stresses and strains are observed. Stresses obtained for all cases with low depth to the tunnel axis (Z₀) are above TMS 602-13 limit of 80 psi.

The construction of new underground facilities inevitably disturbs the soil above and will have an impact on the existing structures within the zone of influence of the excavation. Adjusting the face pressure of TBM to reduce the volume loss shall be considered to reduce displacements.

In cases where mitigation measures cannot be postponed, relining sections will be exposed to maximum tensile stresses should be considered. In extreme cases permeation grouting shall be used as ground improvement mitigation measure.

This study used a constant volume loss of 0.5 %. For future work the impact of volume loss can be investigated on the settlement trough and induced stresses.

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