

ISTANBUL TECHNICAL UNIVERSITY ★ GRADUATE SCHOOL OF SCIENCE
ENGINEERING AND TECHNOLOGY

**BUILDING RESPONSE TO GROUND MOVEMENTS INDUCED BY
TUNNELLING AND EXCAVATION**



Ph.D. THESIS

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Department of Civil Engineering

Structural Engineering Program

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**TÜNEL VE ŞAFT KAZILARINDAN KAYNAKLANAN ZEMİN
HAREKETLERİ KARŞISINDA BİNALARIN DAVRANIŞI**

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To my wife and son Toprak,



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ABBREVIATIONS

CE	: Centre Walkway Tunnel Excavation
CNVs	: Short Connection Excavations
DEM	: Discrete Element Method
FEM	: Finite Element Method
GT	: Green-field Settlement Measurement Point
ISL	: Inclined South Entrance Tunnel
ITU	: Istanbul Technical University
LTSM	: Limiting Tensile Strain Method
NATM	: New Austrian Tunneling Method
NE	: Northeast
NPF	: North Platform Tunnel
NPFV	: North Ventilation Chamber
NPLT	: North Pilot Tunnel
NW	: Northwest
PU	: Polyurethane
RC	: Reinforced Concrete
SE	: Southeast
SB	: Settlement Measurement Bolt
SPF	: South Platform Tunnel
SPFV	: South Ventilation Chamber
SPLT	: South Pilot Tunnel
SW	: Southwest
WVS	: West Ventilation Shaft
3D	: Three Dimensional
2D	: Two Dimensional



SYMBOLS

δS_v	: Differential settlement
$S_{v,A}, S_{v,B}$	: Settlement at A and B points
β	: Angular distortion
β_{max}	: Maximum value of angular distortion
Δ/L	: Deflection ratio
Δ_{sag}/L_{sag}	: Deflection ratio in sagging
Δ_{hog}/L_{hog}	: Deflection ratio in hogging
$(\Delta/L)_{meas}$	: Deflection ratio calculated from the case data
ω	: Rigid rotation of building
φ	: Slope of the ground settlement profile
ε_{crit}	: Critical tensile strain
ε_{lim}	: Limiting tensile strain
$\varepsilon_{lim(low)}$	: Lower boundary of limiting tensile strain
$\varepsilon_{lim(up)}$	: Upper boundary of limiting tensile strain
E	: Elasticity modulus of represented building or wall
G	: Shear modulus of represented building or wall
E/G	: Longitudinal/shear stiffness ratio of building or wall
I	: Second moment of the area of the fictitious deep beam
P	: Fictive central point load
k	: Shear coefficient
ε_{bmax}	: Maximum value of tensile strain due to bending
ε_{dmax}	: Maximum value of diagonal tensile strain
ε_h	: Average horizontal strain
$S_{h,A}, S_{h,B}$	: Horizontal movement at A and B points
ΔS_h	: Differential horizontal movement
ε_t	: Total tensile strain
t	: Neutral axis depth of fictitious beam
θ_{max}	: Crack orientation measured as the angle between the plane on which ε_t acts and the vertical plane

C	: Coefficients corresponding to the equations of fictitious beam
C_b	: Coefficients corresponding to the equations of fictitious beam in bending
C_b^*	: Adapted coefficient forms of C_b when horizontal strain is considered
C_d	: Coefficients corresponding to the equations of fictitious beam in diagonal tension
C_d^*	: Adapted coefficient forms of C_d when horizontal strain is considered
C_{low}	: Back-calculated lower value of coefficients corresponding to the equations of fictitious beam
C_{up}	: Back-calculated upper value of coefficients corresponding to the equations of fictitious beam
ν	: Poisson's ratio
δ_{vm}	: Maximum ground settlement behind an excavation wall
E_1, E_2, E_{mi}	: Young's modulus of masonry
K_n, K_s	: Bedding stiffness in normal and tangential direction
k_{czi}	: Crack interface stiffness in normal direction
k_{cxi}, k_{cyi}	: Crack interface stiffness in two tangential directions
k_{bzi}	: Bedding interface stiffness in normal direction
$\varepsilon_1, \varepsilon_p$	: Principal tensile stress
$\varepsilon_{1,max}, \varepsilon_{pmax}$	: Maximum value of principal tensile stress
w, h	: Width and height of door and window openings
E_s	: Elastic modulus of soil
γ_s	: Volumetric weight of soil
γ_m	: Unit weight of masonry
φ	: Angle of internal friction
L_f	: The length of fictitious masonry tie
$\delta_v(x,y)$	: Applied vertical prescribed deformation
$\delta_{wall}(x,y)$	: Resultant vertical displacement at the bottom of the walls
q_a	: Allowable bearing pressure
δ_{cz}	: Relative crack displacement
H_e	: The depth of excavation
H_i	: The settlement influence depth below the excavation level
r	: Distance from the excavation wall
W	: Settlement trough width by Caspe
B	: Excavation width
δ_v	: Ground settlement at any distance r from the excavation wall

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BUILDING RESPONSE TO GROUND MOVEMENTS INDUCED BY TUNNELING AND EXCAVATION

SUMMARY

In urbanized regions, two important infrastructure activities; subway projects and deep foundation excavations can cause significant negative effects on neighboring surface structures. The increase in demand for transportation, residential and office buildings in high-density areas increases the number of such infrastructure investments. Since lateral urban growth is limited in these areas, tall structures with deep foundation pits have been built to use vertical spaces efficiently. As a result, vertical and horizontal ground movements due to tunneling or deep excavation activities pose a serious risk of damage to existing surface buildings and infrastructure services.

Some characteristic building damages that can arise from tunnelling- and excavation-induced ground movements are classified in the literature as cracking, jammed doors, failure of beams, defects of some sensitive devices, and service pipes. In most cases, the first cracks appear around the corners of the window and door openings; since these are the most vulnerable parts of the wall. Later, as the differential effects of the settlement increase, they can multiply and expand. Severe vertical cracks in the upper part of the masonry walls and subsequent separation along these cracks have also been observed in some cases. Damage caused by tunnelling and excavation can cause severe loss of strength and stiffness.

In order to determine an effective protection method to prevent or mitigate building damage, anticipated magnitude and distribution of tunneling and excavation-induced ground movements must be accurately estimated, and then building response should be predicted. However, this bi-directional problem that involves excavation, soil and building is often not easily solvable due to the uncertainties in material properties, excavation details, construction steps and soil-structure interactions and the difficulties in modelling all these components and the interactions between the ground and the building.

In practice, the risk of building damage from tunneling- and excavation-induced ground movements is assessed at three levels: (i) preliminary, (ii) second-stage, and (iii) detailed assessment. At the preliminary level, the risk is assessed against the limit values of the maximum settlement and rotation of the building, which have been specified based on the previous empirical observations. In the second-stage assessment, an elastic beam analogy is used for buildings and an analytical solution is performed. As the risk level is determined by comparing with the pre-defined empirical strain ranges, the second-stage risk assessment is also based on a quasi-experimental technique. This method, which is formed using the elastic beam approach and uses the pre-defined limit values of the tensile stresses, is also called the limiting tensile strain method (LTSM). At the preliminary and second-stage assessments, building is assumed to follow the deformed ground profile without any relative movement in-between. During the detailed assessment level, a comprehensive examination is carried out using modern numerical tools such as finite element method

(FEM). During, such an analysis, the non-linear properties of the materials (for soil and building), the soil-structure interaction, and the construction stages for the excavation or tunnel can be taken into account. Of course, this latter method offers the most realistic results, but it is apparently more complex, time consuming and costly. Modelling also requires extensive data.

According to the building risk assessment procedure; generally, buildings with moderate or high risk at the preliminary assessment level are re-evaluated at the second-stage assessment level. Similarly, buildings with moderate or high risk at the second-stage assessment level are re-evaluated at the detailed assessment stage. The final risk level is achieved by following these steps. In a typical influence zone, for only few of the buildings and for important historic buildings, detailed assessments are found necessary. The previous experiences showed that for the majority of the buildings the final risk decision is usually made at the second-stage assessment level using the LTSM. Therefore, if the accuracy and precision of the LTSM used in the second assessment stage is increased, not only the accuracy of the final risk decisions determined by this method will increase but also the number of buildings to be evaluated using more costly and more time consuming methods during the detailed risk assessment will be reduced.

Due to the simplicity of the method, LTSM offers rapid evaluation. However, this situation causes some important factors such as soil-structure interaction to be ignored. Or it has been shown, for example, that the damage class of the masonry buildings with large openings in the load-bearing walls are underestimated due to the inability of this method to consider stress concentrations.

Although it is often used in practice, the number of studies in which the LTSM has been analyzed using the real site data is still limited. In addition, the number of projects where the project information is recorded in detail in a way that allows an in-depth analysis of this method is very few.

After a brief introduction (Chapter 1), this study first focuses on the existing methods which dealt with the prediction of the building damage caused by tunneling or deep excavations. A very extensive and comparative literature review is presented for them to convey the current state-of-the-art. Reported advantages and disadvantages of the LTSM are clearly addressed (Chapter 2).

Chapter 3 is devoted to the development of a more independent sight for the LTSM that will ease the interpretation of the relationships between deflection ratio, structure's geometry, longitudinal/shear stiffness ratio and the limiting tensile strain. For this purpose, a transfer function view is proposed. Another original contribution is the inclusion of the effect of average horizontal strain in the modified beam equations. It used to be considered separately in the original LTSM. Using reported data and observed damage classes of real and simulated case studies available in the literature, back-calculations for the coefficients of the transfer function are made. The results showed that the observed damage and measured crack widths are more or less compatible with the limiting tensile strain boundaries. However, for the cases in which moderate or higher damage was observed due to excavations, the use of the LTSM results in underestimation of the resultant damage class. Also, it was shown that the use of the LTSM can cause underestimated damage predictions for the masonry buildings with large openings.

In Chapter 4, a detailed examination of a case, in which a masonry structure has been affected by the ground movements caused by a deep shaft and tunnel excavations, is

presented. The structure has a historical importance and possesses several pre-existing cracks in the load-bearing walls. It is shown that the LTSM underestimates the damage class due to presence of large openings in the load-bearing walls. This case is also a good example for which several information gaps and uncertainties are demonstrated along with the considered assumptions and simplifications. The objective is to demonstrate the added value of a novel two-stage numerical analysis approach for the assessment of the settlement response of the building. The first stage comprises analyses of the structural behaviour using the monitored settlements for each wall. The second stage examines the behaviour of the complete building as a whole. The effects of soil-structure interaction and the pre-existing cracks are considered through discrete interface elements. The parametric analyses executed in the second stage also lead to generalizable results of potential use beyond the specific case. The analysis results showed that using a two-stage analysis approach, the actual response of the building can be satisfactorily represented. Including existing cracks into modelling through the discrete interfaces reduces tensile strains in the close vicinity but results in an increase in the tensile strains of other neighbouring sections. Excluding both bedding and existing crack interfaces during modelling results in more severe cases in terms of tensile strain levels and a higher concentration of tensile strains. As generalizable results of this numerical study, previous findings in the literature was also confirmed: Increased structural stiffness through the increase of the Young's modulus of the masonry walls reduces the conformity of the structure to the applied prescribed settlements and leads to lower tensile strains. Increase of the soil stiffness is simulated with the increase of the bedding stiffness. The higher the bedding stiffness is the more the structure conforms to the applied prescribed settlements. This eventually results in increased tensile strains. As an extreme condition, excluding bedding interface corresponds to a case with infinite bedding stiffness.

In Chapter 5, a holistic conclusion of the findings and suggestions for the future studies are presented.



TÜNEL VE ŞAFT KAZILARINDAN KAYNAKLANAN ZEMİN HAREKETLERİ KARŞISINDA BİNALARIN DAVRANIŞI

ÖZET

Kentleşmiş bölgelerde, iki önemli altyapı faaliyeti; metro projeleri ve derin temel kazıları, komşu yüzey yapıları üzerinde önemli derecede olumsuz etkilere neden olabilir. Yüksek yoğunluklu bu bölgelerde ulaşım, konut ve ofis binalarına olan talebin artması, bu tür altyapı yatırımlarının sayısını artırmaktadır. Bu alanlarda, yatay kentsel büyüme olanağı sınırlı olduğundan, düşey alanları daha verimli kullanmak için derin temel çukurları olan yüksek binalar inşa edilmektedir. Sonuç olarak, tünel açma veya derin kazı faaliyetleri nedeniyle meydana gelen düşey ve yatay zemin hareketleri, mevcut yüzey yapıları ya da su iletim boruları gibi altyapı hizmetleri üzerinde ciddi hasar riskleri oluşturmaktadır.

Literatürde, tünel açma ve kazı kaynaklı zemin hareketlerinden kaynaklanan sorunlar, genel olarak, bina duvarlarında ve taşıyıcı sistem üzerinde çatlaklar, kapıların ve pencerelerin sıkışması, kirişlerin taşıma gücünü yitirmesi, bazı hassas cihazların arızalanması ve altyapı hizmetlerinin hasar görmesi olarak sınıflandırılmaktadır. Çoğu durumda, gerilme yığılmaları bakımından elverişsiz pencere ve kapı açıklıklarının köşelerinde, farklı oturmalarından kaynaklanan çatlaklara rastlanmaktadır. İlerleyen aşamalarda, farklı oturumların büyüklüğü arttıkça, bu çatlakların boyu uzamakta veya genişleyebilmektedir. Zemin profilinin dışbükey bölümü üzerinde bulunan yığma taşıyıcı duvarlara sahip binalarda, üste yakın kesitlerde önemli düşey çatlakların ortaya çıktığı rapor edilmiştir. Oturmaların artması ile bu çatlaklar boyunca meydana gelen ayrılmalar gözlemlenmiştir. Tünel açma ve kazıdan kaynaklanan hasarlar, yapı dayanımının ve rijitliğinin ciddi şekilde azalmasına neden olabilir.

Bina hasarını önlemek veya hafifletmek için etkili koruma yöntemlerinin belirlenmesi amacıyla, tünel açma ve kazı kökenli zemin hareketlerinin büyüklüğünün ve dağılımının doğru olarak belirlenmesi ve daha sonra bu etkiler altında yapı davranışının tahmin edilmesi gerekmektedir. Ancak, bu problem, zemin ve yapı arasında karşılıklı bir etkileşim problemidir ve çoğu kez malzeme özellikleri, kazı detayları, inşaat aşamaları gibi konulardaki belirsizlikler ve bilinmeyenlerden dolayı kolaylıkla çözümlenemeyebilir.

Uygulamada, tünel açma ve kazı kaynaklı zemin hareketlerinden kaynaklanan bina hasar riski üç seviyede değerlendirilir: (i) ön seviye, (ii) ikinci seviye ve (iii) detaylı değerlendirme. Ön seviye risk değerlendirmesi, ampirik gözlemlere dayalı olarak belirlenen oturma ve dönme sınır değerlerinin, her bir binanın izdüşümü üzerinde hesaplanan en büyük oturma ve dönme değerleri ile karşılaştırılması yoluyla belirlenir. İkinci aşama değerlendirmede, binalar için bir kiriş benzeşimi kullanılır. Kiriş denklemlerinin analitik olarak çözülmesi ile en büyük çekme şekildeğiştirmesi hesaplanır. Risk seviyesi, önceden ampirik olarak belirlenmiş çekme şekildeğiştirmesi değerleri ile hesaplanan değerlerin karşılaştırılmasıyla belirlenir. Bu nedenle, ikinci aşama risk değerlendirmesi de yarı ampirik bir tekniğe dayanmaktadır. Elastik kiriş yaklaşımı kullanılarak oluşturulan ve çekme şekildeğiştirmelerinin önceden

tanımlanmış sınır değerlerini kullanan bu yöntemde sınır çekme şekildeğiřtirmeleri yöntemi de (SÇŞY) denilmektedir. Ön ve ikinci seviye deęerlendirmede, binanın, aralarında herhangi bir görelî hareket olmaksızın oturma yapmış zemin profilini takip ettięi varsayılmaktadır. Üçüncü deęerlendirme aşamasında, sonlu elemanlar yöntemi gibi modern sayısal araçlar kullanılarak ayrıntılı bir inceleme yapılır. Malzemelerin (zemin ve bina için) doğrusal olmayan özellikleri, yapı-zemin etkileşimi, kazı ya da tünel için yapım aşamaları dikkate alınabilir. Daha karmaşık, zaman alıcı ve maliyetli olsa da, bu deęerlendirme yöntemi sayesinde gerçeęe daha yakın çözüme ulaşılabılır. Ayrıca, modelleme yapabilmek için daha fazla bilgiye ihtiyaç duyulur.

Bina risk deęerlendirme prosedürüne göre; genel olarak, ön deęerlendirme seviyesinde risk seviyesinin orta veya yüksek olduęu binalar ikinci aşama deęerlendirme düzeyinde yeniden deęerlendirilmektedir. Benzer şekilde, ikinci aşama deęerlendirme seviyesinde risk seviyesinin orta veya yüksek olduęu binalar ise, detaylı deęerlendirme aşamasında yeniden deęerlendirilmektedir. Nihai risk seviyesini bulana kadar bu adımlar takip edilir. Tipik bir kazı etki bölgesi içinde, çoęu zaman, sadece birkaç bina ve tarihi binalar için detaylı deęerlendirmeler yapmak gerekli olmaktadır. Önceki deneyimler, binaların çoęunluęu için nihai risk kararının genellikle SÇŞY kullanılarak ikinci aşama deęerlendirme seviyesinde yapıldığını göstermiştir. Bu nedenle, ikinci deęerlendirme aşamasında kullanılan SÇŞY'nin doğruluęu ve kesinlięi artırılabilirse, sadece bu yöntemle belirlenen nihai risk kararlarının doğruluęu artmakla kalmaz, aynı zamanda detaylı deęerlendirmede kullanılan daha masraflı ve daha fazla zaman gerektiren yöntemler ile deęerlendirilecek binaların sayısı da azalacaktır.

Kullanılan yöntemin basitlięi nedeniyle, SÇŞY hızlı deęerlendirme olanaęı sunmaktadır. Ancak bu durum yapı-zemin etkileşimi gibi bazı önemli faktörlerin dikkate alınamamasına da neden olmaktadır. Ya da örneğin, SÇŞY taşıyıcı duvarlarında geniş boşluklar bulunduran yığma binaların hasar durumunu gerilme yığılmalarını dikkate alamadığını için olduğundan daha az olarak tahmin edebilmektedir.

Uygulamada sıklıkla kullanılmasına rağmen, SÇŞY'nin gerçek saha verilerini kullanarak yeniden gözden geçirildięi çalışmaların sayısı hala sınırlıdır. Ayrıca, saha verilerinin ayrıntılı bir biçimde analiz edilmesine imkan verecek şekilde kaydedildięi projelerin sayısı da çok azdır.

Kısa bir giriş bölümünün ardından (1. Bölüm), bu tez kapsamında ilk olarak, tünelcilik faaliyetlerinden veya derin kazılardan kaynaklanan bina hasarlarının tahmin edilmesi için geliştirilmiş mevcut yöntemler ele alınacaktır. Bu alanda yapılmış çalışmalar kapsamlı ve karşılaştırmalı bir literatür taraması ile sunulmuştur. Ayrıca, SÇŞY'nin mevcut literatürde belirtilen avantaj ve dezavantajları da açık bir biçimde ortaya konulmaktadır (2. Bölüm).

3. Bölüm'de, SÇŞY'de kullanılan sehim oranı, yapı geometrisi, yapının uzama ve kayma rijitlikleri oranı ve sınırlayıcı çekme şekildeğiřtirmesi terimlerinin arasındaki ilişkilerin yorumlanmasını kolaylaştıracak daha bağımsız matematiksel bir ifadenin geliştirilmesi amaçlanmıştır. Bu bağlamda, transfer fonksiyonlarından faydalanılmıştır. Ek olarak, zemin ortalama yatay şekildeğiřtirmesinin etkisi modifiye edilmiş derin kiriş denklemlerine dahil edilmiştir. Orijinal SÇŞY'de zemin yatay şekildeğiřtirmelerinin etkisi doğrudan dikkate alınmamaktadır. Literatürde mevcut olan gerçek ve simüle edilmiş vaka çalışmalarından elde edilen veriler ve gözlenen hasar sınıfları kullanılarak, transfer fonksiyonunun katsayıları için geri hesaplamalar yapılmıştır. SÇŞY'nin, pratik bir araç olarak, kısıtları göz önünde bulundurulduğunda,

kazı etki alanında bulunan binalar için yapılacak hızlı değerlendirmeler için kullanışlı olduğu sonucuna varılmıştır. Ancak, mevcut deneysel ve sayısal çalışmaların bulguları, SÇŞY'nin basitliği nedeniyle gerçek bina davranışını öngörmede büyük ölçüde eksik olabileceğini göstermiştir. Sonuçlar, ayrıca, gözlenen hasarın ve ölçülen çatlak genişliklerinin sınır çekme şekildeğiştirmesi değerleri ile belirli oranda uyumlu olduğunu göstermiştir. Orta ya da daha yüksek hasarın gözlemlendiği durumlarda, orijinal derin kırıış denklemlerinin, ortaya çıkan hasar sınıfını daha düşük bir değer olarak dikkate alma eğiliminde olduğu ortaya koyulmuştur. Ayrıca, SÇŞY'nin kullanımının büyük duvar boşluklarına sahip yığma yapılarda muhafazakar olmayan hasar tahminlerine yol açabileceği gösterilmiştir.

4. Bölüm, derin bir şaft ve tünel kazılarının neden olduğu zemin hareketlerinden etkilenen yığma taşıyıcı sisteme sahip tarihi bir yapı için yapılan ayrıntılı analizleri ve sonuçlarını kapsamaktadır. Yapının taşıyıcı duvarlarında kazı öncesi dönemde ortaya çıkmış çatlaklar bulunmaktadır. Yapının duvarlarında bulunan geniş pencere ve kapı boşluklarından dolayı, SÇŞY'nin, kazılardan dolayı oluşması muhtemel hasarları olduğundan daha düşük seviyede tahmin ettiği gösterilmiştir. Bu çalışma ayrıca, dikkate alınan varsayımlar ve basitleştirmelerle birlikte saha raporlarından elde edilemeyen bilgi eksikliklerinin ve belirsizliklerin de gösterildiği iyi bir örnek vaka durumu teşkil etmektedir. Bu bölümde literatürdeki diğer vaka çalışmalarından farklı olarak, iki aşamalı bir nümerik analiz yaklaşımı önerilmiştir. İlk aşamada, her bir duvar, sahada izlenen oturma değerleri kullanılarak ayrı ayrı analiz edilmiştir. Bu şekilde, oturma profili ile ilgili belirsizlikler geçici olarak devre dışı bırakılarak yapı modeli kalibre edilmiştir. İkinci aşamada, bina davranışı bir bütün olarak incelenmiştir. Yapı-zemin etkileşiminin ve önceden var olan çatlakların etkileri de bir takım arayüz elemanları kullanılarak modellemede dikkate alınmıştır. İkinci aşamada yapılan parametrik analizler ayrıca, incelenen vakanın ötesinde genelleştirilebilir sonuçların elde edilmesini sağlamıştır. Analiz sonuçları, iki aşamalı bir yaklaşımın kullanılması ile binanın oturma etkileri altındaki gerçek davranışının tatmin edici bir şekilde simüle edilebileceğini göstermiştir. Elde edilen gerilmelerin büyüklüğü ve modellenen mevcut çatlakların davranışı, bu yapı için ölçülen ve gözlemlenen bilgiler ile uyumludur. Parametrik analiz sonuçları, zemin / yapı rijitlik oranı arttıkça, yapının oturma etkileri altındaki kırılabilirliğin arttığını ortaya koymuştur. Ayrıca, yapı-zemin etkileşiminin analizlerde dikkate alınması durumunda farklı oturmalarından kaynaklanan gerilmelerin azaldığı gözlemlenmiştir. Mevcut çatlakların analizlere dahil edilmesi ile bu çatlakların çevresindeki gerilmelerin azaldığı, ancak komşu kesitlerdeki gerilmelerin arttığı gözlemlenmiştir.

5. Bölümde, elde edilen sonuçlar bütünsel bir bakış açısıyla sunulmuş ve incelenen konu kapsamında gelecekte yapılabilecek çalışmalar için önerilerde bulunulmuştur.



1. INTRODUCTION

1.1 Background

In urbanized regions, two important infrastructure activities; subway projects and deep foundation excavations can cause significant negative effects on neighboring surface structures by inducing vertical and horizontal ground movements at their foundations. According to Boscardin (1980), the main causes of excavation-induced ground movements are an alternation in the state of stress within the ground, ground loss or soil removal, and change in the groundwater regime. In addition, the construction technique, excavation size, soil type and stiffness, stiffness of excavation support and support spacing, delayed support and grouting installation, and face stability (in tunnels) are other significant parameters affecting the magnitude of tunnelling- and excavation-induced ground movements (Son, 2003).

The increase in demand for transportation, residential and office buildings in high-density areas increases the number of such infrastructure investments. In Istanbul with 15 million population, the total length of subway projects which are expected to be completed between the years 2018 and 2019 is 122 km and also new subway projects with total length of about 400 km is planned to be finalized after 2019 (Url-1, 2018).

The potential damage risk for thousands of buildings in the influence zone of different tunnel and deep excavation projects around the world must be determined accurately. In this way, repair costs are reduced or avoided, the reputation of the projects is protected in the society and the decreasing of cases and disputes is ensured.

The existing methodology for evaluating building damage risk against tunnelling and excavation-induced ground movements depends on the studies of Rankin (1988) and Mair et al. (1996). According to this; risk is assessed in three stages: (i) preliminary, (ii) second stage, and (iii) detailed.

In the preliminary stage, buildings that are exposed to less than 10 mm settlement (vertical movement of a part of or entire the building due to ground displacement) and/or 1/500 rotation (angular rigid movement of a building with respect to a virtual

horizontal axis) are regarded as negligibly risky. For these buildings, no further risk assessment is needed. Depending on the sensitivity of the building, these values can be further reduced.

Other buildings subjected to the ground movement beyond these limits are evaluated in the second-stage assessment. In the second-stage assessment, a beam analogy for the building walls (the masonry walls or infill walls of frame buildings that are perpendicular to tunnel line or the excavation wall, which is the most unfavorable) is adopted as proposed by Burland and Wroth (1974). Applicability of this approach for both masonry walls which are the monolithic constructions and infilled wall - surrounding beam/column compositions can be justified because of the fact that the infilled frames may respond similarly to masonry structures due to the increased shear stiffness. Son and Cording's (2011) numerical settlement analyses showed that reinforced concrete frames with brick infilled walls had responses similar to those of brick masonry walls up to slight cracking. In subsequent stages, while cracks propagate rapidly in masonry structures, frame elements (beams and columns) usually prevent crack propagation and further damage by confining the infilled walls. Connection properties between infill walls and frame elements are surely highly influential on this mechanism. The settlement effects are accounted for as fictive external loading in the equations to be used to solve the load-deflection problem of the analogous beam. Then, the maximum value of bending and diagonal tensile strains are calculated and, whichever is greater, is compared to predefined limiting tensile strain ranges to determine the damage class and corresponding risk level. This procedure is called Limiting Tensile Strain Method (LTSM).

For buildings of which risk is determined as moderate and higher in the second-stage assessment, detailed examinations should be performed using more sophisticated methods like finite element method (FEM). For example, during the tunnel and shaft excavations at the Sirkeci Metro Station of Marmaray Project (in Istanbul), 135 buildings were examined in the second-stage assessment and 58 (40%) of these buildings (with medium and high risk) were reassessed in the detailed assessment phase. In other words, for 60% of these structures, the final risk decision was made according to the results of the LTSM (Monthly Geotechnical Monitoring Reports, 2008-2013). This shows the importance of the second-stage assessments. Obviously, when the complexity of the evaluation method is increased from the preliminary to the

detailed level, the processing time and related costs also increase. Therefore, if the accuracy and precision of the LTSM used in the second-stage assessments increase, it will not only increase the accuracy of the final risk decisions but also reduce the number of buildings to be evaluated by expensive and time consuming methods i.e FEM in the detailed evaluation stage. Although some improvements have been made i.e. relative stiffness method of Potts and Addenbrooke (1997) to account for the building stiffness and soil-structure interaction in a more realistic way, owing to its simplicity and recognition, LTSM is still only the method suggested in many risk assessment contracts i.e Gayrettepe – 3rd Airport subway project in Istanbul. In order to improve the second stage assessment process, the advantages, disadvantages and limitations of the LTSM should be revealed by verifications made using different case reports from different projects. However, the number of such studies in which LTSM is reviewed comprehensively is rather limited (Son and Cording, 2007 and Netzel, 2009).

For buildings of which moderate and high risk is determined, a detailed examination is usually carried out. The factors that should be taken into account in a detailed assessment include the following (Burland et al., 2011).

- Consideration of the sequence and method of tunnel and excavation construction.
- Consideration of structural details of each building.
- Consideration of foundation type.
- Consideration of the orientation of the buildings and the skewed positioning relative to the tunnel or the excavation, and taking into account the 3D effects of ground movement causing the building to twist.
- Consideration of the soil-structure interaction.
- Consideration of previous movements and effects that might be influential on the current tolerances of the building.

Widespread use of computer technology since the beginning of the 90s has allowed researchers to investigate the single or combined effects of these parameters (Giardina, DeJong & Mair, 2015a Potts & Addenbrooke, 1997; Son & Cording, 2011). Complicated soil-structure interaction, nonlinearities in building materials and soil,

effects of different structural configurations, building location, size and weight effects can be taken into account using numerical methods. Furthermore, the addition of tunnel construction modelling to the finite element analyses allowed the investigation of coupled effects of building-tunnel-soil behaviour, even in three dimensions (3D) (Amorosi, Boldini, de Felice, Malena & Sebastianelli, 2014; Augarde, Burd & Houlsby, 1998; Franzius, 2003; Jenck & Dias, 2004; Pickhaver, Burd & Houlsby, 2010; Son, 2015).

Each case to be assessed is unique and requires a special treatment. Therefore, a general guideline cannot be given. Moreover, due to inherent uncertainties of the problem and lack of information, in many cases, detailed assessment requires experience and engineering judgement. Several assumptions regarding soil, foundation, ground movements and building might have to be made. Well-documented case studies in which different types of excavation, soil and building conditions are examined offer a great resource to understand this complicated problem.

1.2 Scope of the Research

This thesis study aims to:

1. review the LTSM and verify the accuracy of the predicted building damage by utilizing the available real site data from the existing studies in the literature: A transfer function view of the fictitious deep beam equations is provided to offer a more independent sight for the interpretation of the relationships between deflection ratio, structure's geometry, longitudinal/ shear stiffness ratio and the limiting tensile strain. In addition, the effect of average horizontal strain is included simply in the modified deep beam equations.
2. demonstrate a challenging case study with project uncertainties and unknowns and utilize the case data for detailed analyses. The LTSM underestimates the building damage. Following the preliminary numerical analyses and conventional evaluations, a novel two-stage numerical approach is proposed in order to put off some of these uncertainties. Furthermore, a parametric study is also conducted for varying properties of soil-structure interaction and building material. This chapter presents important considerations in showing how typical problems of such case studies are tackled.

1.3 Outline of the Thesis

This Ph.D. thesis consists of five main chapters.

Chapter 1 addresses to a brief background of the current building risk assessment strategy, scope of the research and outline of the thesis.

Chapter 2 provides an extensive literature review of the methods evaluating the building response to tunnelling- and excavation-induced ground movements.

Chapter 3 specifically focuses on the LTSM and presents a novel transfer function view for the original elastic beam functions.

In Chapter 4, the settlement response of a masonry mosque structure to a shaft excavation and tunnelling activities is numerically examined. The structure has a historical importance and possesses several existing cracks in the load-bearing walls.

In Chapter 5, a holistic conclusion of the findings and suggestions for the future studies are presented.



2. LITERATURE REVIEW

2.1 Nature of the Ground Movements due to Tunnel and Deep Excavations

During a tunnel excavation, induced ground settlements form a trough shape perpendicular to the tunnel line (Figure 2.1(a)). If the ground surface is free of buildings, this shape is called the free-field ground settlement profile. Peck (1969) proposed to use Gaussian Normal Distribution Curve functions to calculate the free-field ground settlement profile in case of open shield tunnel excavations in clay soils. Although Peck's method has been widely accepted, some discrepancies have also been reported regarding the use of this method with different soil types and excavation techniques (Laefer & Kim, 2004; Marshall et al., 2012). Standing & Selemetas (2013) stated that numerical methods can provide better estimates for tunnelling-induced free-field ground settlements in the cases where the complex mechanisms take place. The authors suggested to incorporate the key variables such as face and tail-skin grouting pressures into the numerical analyses. Mathew and Lehane (2012) highlighted the influence of chosen soil model and parameters on the numerical free-field ground settlement predictions. In addition, they observed that modelling of tunnel constructions by imposing volumetric contraction might not reflect all of the complexities associated with actual tunnelling procedures accurately.

Ongoing tunnel excavations result in advancing ground settlements in the longitudinal direction of the tunnel line. Depending on the position of tunnel face, the longitudinal ground movements can be calculated using the Cumulative Probability Function. Furthermore, both in perpendicular and longitudinal direction, significant horizontal movements can occur. Horizontal strains can be calculated by taking the derivative of the function used for the horizontal ground movements.

In a tunnelling-induced ground settlement profile, while the concave part of the profile is referred to as sagging, the convex part is referred to as hogging. Buildings can be situated in different positions with respect to tunnel line and they can modify ground settlements depending on their restraint capabilities (size, weight, foundation type,

etc.). In Figure 2.1 (a), different buildings that are sagging (i), in the vicinity of the inflection point (ii) and hogging (iii) are schematically illustrated (settlement modification is not shown in Figure 2.1(a)). In this figure, e refers to the eccentricity between the centre of the building and the tunnel centre.

Deep excavations are influential in a lateral distance ranging from two times to four times the excavation depth (Peck, 1969; Clough & O'Rourke, 1990). Several empirical and semi-empirical models have been proposed to predict the free-field ground settlement profile due to adjacent deep excavations (Peck, 1969; Clough & O'Rourke, 1990 and Hsieh & Ou, 1998). Hsieh and Ou (1998) introduced possible ground settlement profiles as shown in Fig 2.1 (b). The authors proposed two different methods to calculate spandrel and concave ground settlement profiles based on the estimated wall deformation.

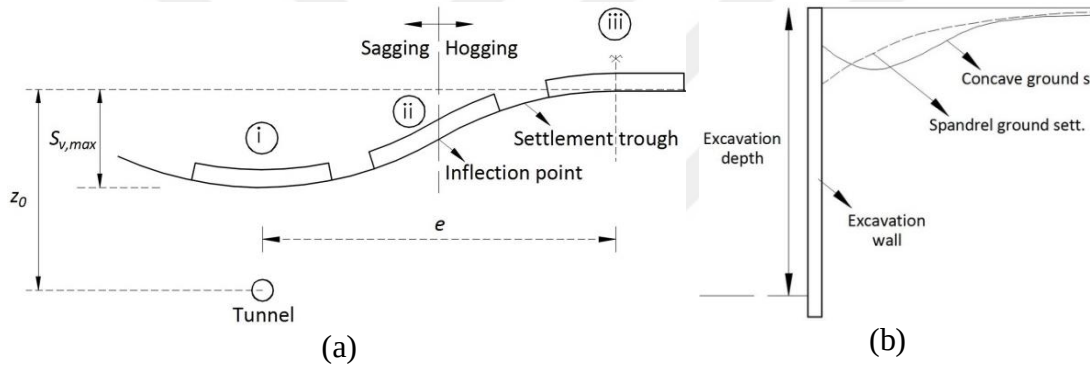


Figure 2.1 : (a) Different positions of buildings in the transverse settlement trough: (i) sagging, (ii) at an inflection point and (iii) hogging. (b) Possible ground settlements due to deep excavations (modified from Hsieh and Ou, 1998).

2.1.1 Calculation of free-field ground movements due to tunneling using Gaussian Normal Distribution Curve Method (Peck, 1969)

Peck (1969) proposed that tunnelling-induced free-field vertical ground movements $S_v(x)$ in the transverse direction (perpendicular to the tunnel line) can be represented by a Gaussian normal distribution curve function (Figure 2.2). In the literature, the free-field surface profile in the transverse direction is usually called a settlement trough.

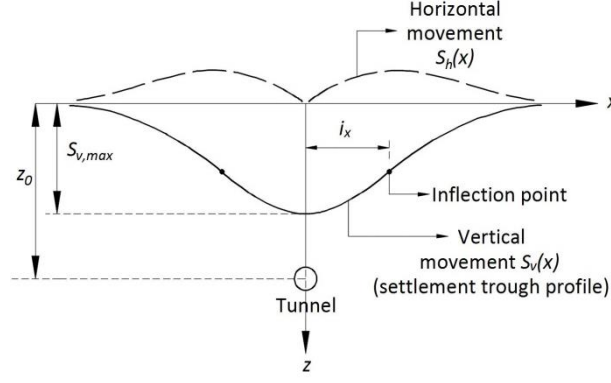


Figure 2.2 : Free-field vertical and horizontal ground movements in the transverse direction (modified from Giardina, 2013).

Using this function, vertical ground movements at any immediate or near-surface point within the profile can be calculated as shown in equation 2.1, where $S_{v,max}$ is the maximum vertical movement right above the tunnel axis, x is the horizontal distance between the vertical axis and the point where the vertical movement is sought, and i_x is the horizontal distance between the vertical axis and the inflection point (Figure 2.2). Parameter i_x thus controls the trough width. O'Reilly and New (1982) proposed a linear expression for the calculation of i_x as given in equation 2.2.

$$S_v(x) = S_{v,max} e^{-\frac{x^2}{2i_x^2}} \quad (2.1)$$

$$i_x = K(z_0 - z) \quad (2.2)$$

In equation 2.2, K is a constant depending on the soil type. K varies between 0.2 and 0.3 for granular soils and 0.4 and 0.5 for stiff clays and is as high as 0.7 for soft silty clays. z_0 is the tunnel depth, and z is the vertical coordinate of the point where the trough width is sought. For $z = 0$, i_x is calculated at the surface. As a general rule, for tunnels in clay strata, the trough width at the surface can be taken as three times z_0 (Burland, Standing, & Jardine, 2001). The volume of the settlement trough per unit length can be calculated using the integration given in equation 2.3.

$$V_s = \int_{-\infty}^{+\infty} S_v(x) dx = \sqrt{2\pi} S_{v,max} i_x \quad (2.3)$$

The volume loss (V_L) is considered a unitless measure of the total ground disturbance (Franzius, 2003). It is calculated using equation 2.4, where D denotes the tunnel outer

diameter. V_L is usually presented as a percentage. For instance, studies conducted in the United Kingdom revealed that V_L can be considered as being between 1% and 3% for shield tunnelling (excavated with a tunnel boring machine, TBM) (O'Reilly & New, 1982) and between 1% and 1.5% for the New Austrian Tunneling Method (NATM) (New & Bowers, 1994). Although the magnitudes of vertical and horizontal ground movements are known to be affected by the excavation technique, the form of settlement trough (Gaussian distribution curve type) is assumed to be independent of the excavation technique (New & Bowers, 1994).

$$V_L = \frac{V_s}{\frac{\pi D^2}{4}} \quad (2.4)$$

Using equations 2.3 and 2.4, $S_{v,max}$ can now be calculated as:

$$S_{v,max} = \frac{V_L \pi D^2}{4\sqrt{2\pi}i_x} \cong \frac{V_L D^2}{3.19i_x} \quad (2.5)$$

The free-field vertical movement at any point within the settlement trough can be associated with V_L as:

$$S_v(x) = \sqrt{\frac{\pi}{2}} \frac{V_L D^2}{4i_x} e^{-\frac{x^2}{2i_x^2}} \quad (2.6)$$

O'Reilly and New (1982) showed that the free-field horizontal ground movement $S_h(x)$ can be calculated as shown in equation 2.7, with the assumption that the resultant ground movement vector is towards the tunnel centre. The variation of S_h in the transverse direction is also presented in Figure 2.2.

$$S_h(x) = -\frac{xS_v(x)}{z_0} \quad (2.7)$$

Similarly, ground movements in the longitudinal direction (parallel to the tunnel line) can also be calculated using equations 2.8 and 2.10 proposed by Attewell and Woodman (1982). In equation 2.8, $S_v(y)_{x=0}$ is the vertical ground movement in the longitudinal direction (y direction) for $x = 0$.

$\Phi(y)$ is the normal cumulative probability function.

$$S_v(y)_{x=0} = S_{v,max} \Phi(y) \quad (2.8)$$

$\Phi(y)$ is expressed as:

$$\Phi(y) = \frac{1}{i_y \sqrt{2\pi}} \int_{-\infty}^y e^{-\frac{y^2}{2i_y^2}} \quad (2.9)$$

In this equation, i_y is the trough width parameter of the longitudinal settlement profile. It is generally assumed to be equal to i_x . Horizontal movement in the longitudinal direction (y direction) is calculated using equation 2.10.

$$S_h(y)_{x=0} = S_{h,max} e^{-\frac{y^2}{2i_y^2}} = \frac{V_L D^2}{8z_0} e^{-\frac{y^2}{2i_y^2}} \quad (2.10)$$

Variations in the vertical and horizontal ground movements in the longitudinal direction are illustrated in Figures 2.3(a) and 2.3(b), respectively. It is noted that the vertical ground movement above the tunnel face is equal to half of $S_{v,max}$ (Figure 2.3(a)).

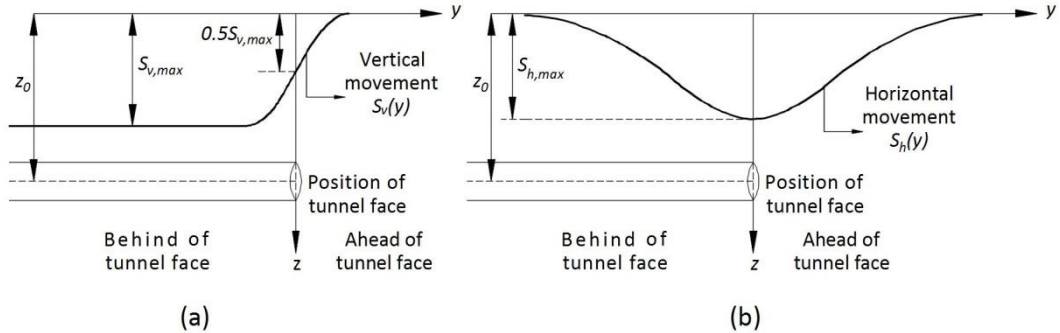


Figure 2.3 : Free-field (a) vertical and (b) horizontal ground movements in the longitudinal direction (modified from Giardina, 2013).

If twin tunnels are present, the calculated ground movements are usually superimposed. Burland (1997) reported that, if the distance between twin tunnels is less than one tunnel diameter, superposition might be unconservative.

2.2 Building Deformation Measures

Tunnelling- and excavation-induced ground movements can damage surface buildings by impinging their foundations. Building deformation measures note measurable geometrical distortions of buildings that are under the influence of tunnelling- and excavation-induced ground movements. They are used to establish correlations between geometrical distortions and building damages. Different building deformation measures have been adopted in the literature. Burland , Broms , & de Mello (1977) observed a wide variety of confusing symbols and terminology used for deformation measures in the literature. It is believed that the complexity is still present, particularly for practitioners and engineers who are not expected to have a complete command of the literature. To calculate building deformation measures, actual site measurements taken from buildings (from the foundation or superstructure) or predicted ground settlement contours provided at the foundation level can be utilized.

Uniform settlements of buildings mostly result in major serviceability problems related to building access and connection to utility lines. On the other hand, structural damage is inevitable in the presence of differential settlements, depending on the straining extent of the settlement (Little, 1969; Peck, Deere, & Capacete, 1956). Building damage may involve structural (cracking, loss of girder bearing, etc.) or nonstructural (jammed doors, malfunction of various sensitive devices and service pipes, etc.) damages (Son & Cording, 2005). Differential settlement refers to a settlement case that results in variable vertical displacements along the building foundation (for continuous or mat foundations) or differential vertical movement in columns or individual footings (mostly for frame structures). Note that the deformation measures described here are only associated with the vertical movement of buildings. The effect of horizontal component of ground movements is discussed separately in the following sections.

2.2.1 Deformation measures for masonry buildings

In Figure 2.4, building deformation measures that have commonly been adopted in previous studies (differential settlement δS_v , angular distortion β and deflection ratio Δ/L) are schematically described considering the potential ground settlement modes (previously shown in Figure 2.1(a)-(b)). For masonry buildings, load-bearing walls are essential elements. Therefore, performance assessments of masonry buildings are

based on the wall damages. Observations show that load-bearing walls perpendicular to a tunnelling line or an excavation wall are damaged more severely compared to the other wall configurations. Therefore, while induced damages are predicted by analytical or empirical methods, only load-bearing walls perpendicular to the tunnelling line or excavation wall are considered.

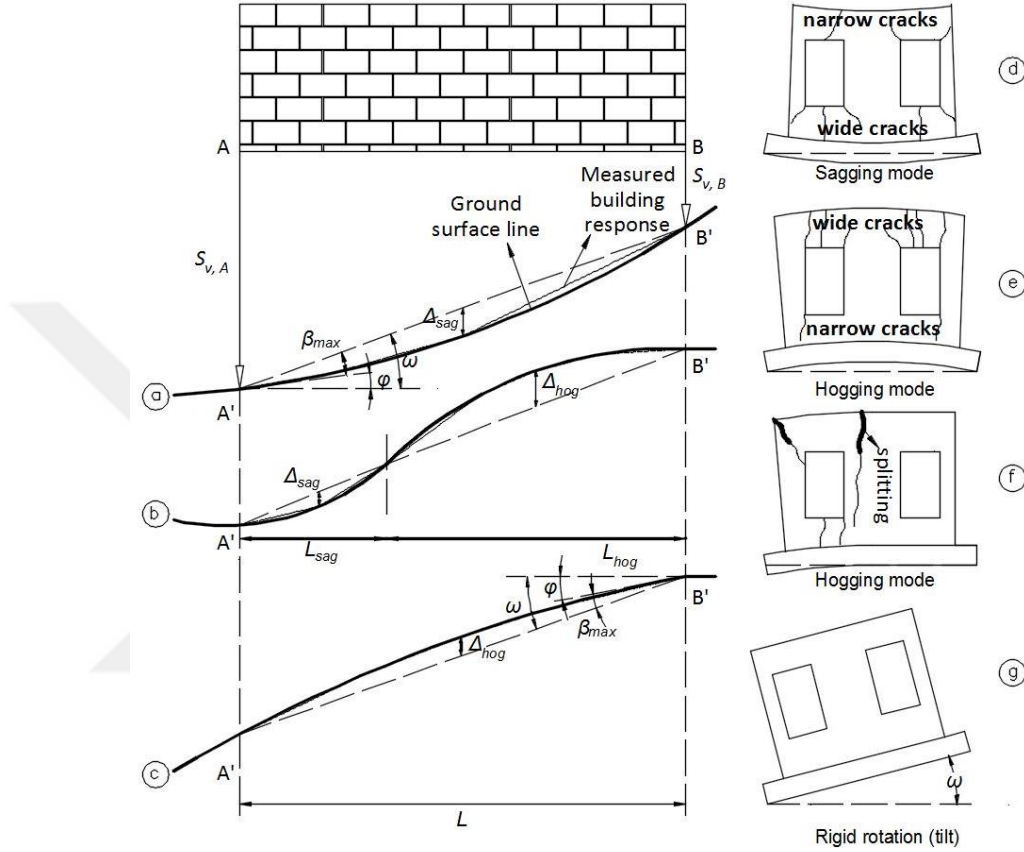


Figure 2.4 : (a) Deformation measures for a masonry wall conforming to a ground settlement profile in (a) the sagging zone, (b) both the sagging and hogging zones and (c) the hogging zone and the typical damages or response (d, e, f and g).

As shown in Figures 2.4(a-c), as the ground settles due to a tunnel excavation, adjacent points A and B on the masonry wall perpendicular to the tunnelling line are displaced to points A' and B' , resulting in straining within the wall. In this case, the differential settlement δS_v between points A and B is calculated using equation 2.11. In this equation, $S_{v,A}$ and $S_{v,B}$ represent the vertical movement of points A and B , respectively.

$$\delta S_v = S_{v,A} - S_{v,B} \quad (2.11)$$

Apart from differential settlement, there are two other important damage measures that are frequently used in the literature: angular distortion (β) and deflection ratio (Δ/L).

In some studies, these two terms are also called relative rotation and relative deflection, respectively. Skempton and MacDonald (1956) were first to introduce the angular distortion term as a damage measure for the investigation of settlement-induced damage. They defined it as the ratio between the differential settlement of two adjacent points on the structure and the distance between these points (between points A and B , it could be $\beta = \delta S_v/L$). Afterwards, Boscardin (1980) modified the definition of β such that the contribution of rigid rotation of the building (ω) (sometimes referred to as tilt) is excluded. It is known that ω corresponds to the free rotation of a building and does not cause any straining within the building (Figure 2.4(g)). In the current literature, β is calculated using the absolute-value expression in equation 2.12 as modified by Boscardin. To calculate β , the other geometrical quantity required is the slope of the ground settlement profile (φ). ω and φ can be measured directly on the site or determined through the predicted site settlement contours. Note that the maximum value of β (β_{max}) should be considered. β_{max} , ω and φ are shown schematically on deformed building foundation lines in Figures 2.4 (a-c).

$$\beta = |\omega - \varphi| \quad (2.12)$$

Polshin and Tokar (1957) proposed the deflection ratio (Δ/L), which is another important damage measure in the literature. Herein, Δ represents the maximum value of the vertical differential distance between the deflected foundation line and the straight line connecting points A and B (Figures 2.4 (a-c)). Δ/L is then obtained by dividing Δ by the corresponding span length L . If the building spans the inflection point (Figure 2.1(a-ii)), it is possible to calculate Δ/L for both the sagging and hogging part of the wall. For these buildings, each part can be considered separately (Mair, Taylor, & Burland, 1996). Δ/L is represented as Δ_{sag}/L_{sag} for sagging and Δ_{hog}/L_{hog} for hogging. Herein, L_{sag} and L_{hog} represent the lengths of the building parts that are sagging or hogging, respectively. There is no consensus in the literature as to which damage measure (β or Δ/L) is more appropriate to detect damage initialization within the building. While some studies adopt β , others use Δ/L . Netzel (2009) compared the use of β and Δ/L . The author proposed using β for calculation of shear-related damage potential and using Δ/L for calculation of bending-related damage potential. Netzel claimed that calculations can otherwise lead to overestimations or underestimations of potential damage. Based on the monitoring

results of different case studies in the UK, Burland, Mair, and Standing (2004) suggested the use of Δ/L because they found it to be less ambiguous than β .

Some typical tunnelling- and excavation-induced building damages are also shown with schematic illustrations in Figure 2.4 (d-f). However, it should be reiterated that damage initialization and propagation are quite complicated and can be affected by numerous different parameters, such as the building length to height ratio (L/H), shear/axial stiffness ratio of the brick-mortar interface in masonry buildings and soil-structure interaction. Furthermore, many researchers have highlighted that cracking due to shear and bending usually occurs simultaneously in actual cases. Apart from these, buildings might tend to experience rigid rotation instead of serious distortion, especially in one or more of the cases where the settlement profile is wide enough, the building height to length ratio is high or the building has a very stiff mat foundation (Figure 2.4 (g)). In these cases, the damage experienced might be quite limited.

2.2.2 Deformation measures for frame buildings

Literature survey results have shown that there is a consensus regarding the reduced vulnerability of steel and reinforced concrete (RC) frame structures compared to masonry structures. This is because frame structures possess a structural continuity, which provides an important advantage compared to masonry structures (Mair, Taylor, & Burland, 1996). Moreover, they usually have rigid RC foundations with high tensile strength and flexural stiffness. Despite their lower vulnerability, settlement effects on the frame structures should not be completely ignored. Meyerhof (1953) showed that, even in the presence of small differential settlement values (up to 12 mm), stresses at the critical sections of beams and columns (due to the self-weight of the building) are doubled. Burland and Wroth (1974), Bryson and Kotheimer (2011), Fu, Yang, Zhang, Klapperich, and Abbas (2014) and Goh and Mair (2014) also presented several case histories in which different extents of tunnelling- and excavation-induced damage of frame structures have been reported. Laefer, Ceribasi, Long and Cording (2009) observed the excavation-induced damages in the beam elements of scaled open frame test specimens.

Within frame structures, frame elements (beams and columns) are found to be far less susceptible to tunnelling-induced cracking than infill walls and panels because they have relatively higher tensile strength. Although the open frames are least vulnerable

by means of their flexibility, infilled frames may respond similarly to masonry structures due to the increased shear stiffness. Son and Cording's (2011) numerical analyses showed that RC frames with brick infilled walls had responses similar to those of brick masonry walls up to slight cracking. This similarity provides the opportunity to use the same deformation measures and calculation methods (i.e., LTSM) for both infilled frame and brick masonry structures. In subsequent stages, while cracks propagate rapidly in masonry structures, frame elements (beams and columns) usually prevent crack propagation and further damage by confining the infilled walls. Connection properties between infill walls and frame elements are surely highly influential on this mechanism.

2.2.3 Proposal of Mair et al. (1996) regarding the calculation of deformation measures

As previously shown in Figure 2.1 (a-ii), buildings can be simultaneously located partially in a sagging and hogging zone. This situation leads to the question of how deformation measures should be calculated. Mair et al. (1996) stated that, in such cases, either part of the building can be assessed separately. Δ_{sag}/L and Δ_{hog}/L are calculated for either part of the building (Figure 2.4 (b)). This was discussed by Finno et al. (2005), who reported that, if separation is performed, an additional shear strain should be included in diagonal tensile strain calculations. Netzel (2009) also examined Mair's proposal using numerical models. The author suggested that separation can be performed provided that the rigid rotation difference between the entire building and the concerned building part does not exceed 15/100.

Mair et al. (1996) also suggested that building parts that experience a vertical settlement movement less than 1 mm do not need to be accounted for in an assessment (Figure 2.1 (a-iii)). They recommended cutting off these building parts and performing calculations considering a reduced span length L . Numerical analyses performed by Netzel (2009) for long buildings showed that Mair's suggestion causes serious underestimations of strains. According to Netzel, for long buildings, building parts beyond the 1-mm line act as support such that the building can be treated as a cantilever beam fixed at the less-settled (under 1 mm) end.

2.3 Methods for Predicting and Assessing Building Damage Resulting from Tunnelling- and Excavation-induced Ground Movements

2.3.1 Empirical methods

Empirical methods are early methods that generate correlations between building damage stemming from ground settlement and deformation measures, such as β and Δ/L , using site observations and simple theoretical relationships. Note that only the vertical components of ground movements are accounted for in these methods because these empirical methods were developed considering settlements under building self-weight. Therefore, only the sagging mode of settlement is considered. Skempton and MacDonald (1956) examined 98 case histories and proposed limitations by correlating observed β values from weight-related settlements and reported building damages (Table 2.1). Those buildings consisted of conventional steel frame, RC frame and masonry structures. Forty out of 98 buildings were apparently damaged by the settlement. As shown in Table 2.1, while a β of 1/300 is defined as the onset of the cracking in panel walls (infill walls of frame structures and masonry load bearing walls), 1/150 is proposed as the lower limit of the severe distortion of the frame structures and severe cracking of masonry walls. β value of 1/500 is suggested not to exceeded as a safety limit.

Table 2.1 : Limiting value of β introduced by Skempton and MacDonald (1956).

Limit value of β	Description
1/750	Limit where difficulties with machinery sensitive to settlements are to be feared
1/600	Limit of danger for frames with diagonals
1/500	Safe limit for buildings where cracking is not permissible
1/300	• Limit where first cracking in panel walls is expected • Limit where difficulties with overhead cranes are to be expected
1/250	Limit where tilting of high, rigid buildings might become visible
1/150	• Considerable cracking in panel walls and brick walls • Safe limit for flexible brick walls, $H/L < 1/4$ • Limit where structural damage of buildings is to be feared • Severe distortion of frame

These limits were then reviewed, and a β value of 1/500 was also confirmed as a safe limit by Bjerrum (1963). Burland et al. (1977) emphasized some important points regarding the use of the study of Skempton and Macdonald (1956). According to these authors, the range of buildings utilized in this study was quite limited. In particular, the number of structural systems with masonry walls was not sufficient. Moreover, the choice of β as a deformation measure that is mostly related to shear distortions (and thus more critical for frame structures) could not completely reflect the actual settlement-induced deformations, which also include bending-related damages (which are more critical for masonry structures).

Three other important concepts used in the calculations, Δ/L , L/H and a critical tensile strain (ε_{crit}), were then introduced by Polshin and Tokar (1957). They put forward some limitations for the Δ/L of buildings based on the site observations and building characteristics, such as the L/H (geometrical) and ε_{crit} (on the material level). ε_{crit} refers to the critical tensile strain that is attained at the onset of visible cracking in load-bearing masonry or infill walls of frame structures. Critical tensile strains of 0.05% and 0.035% were defined for masonry and RC elements, respectively. Note that the choice of L/H as an influential parameter is important because it practically corresponds to a measure of the building stiffness. This approach in which Δ/L , L/H and ε_{crit} were associated would later form the basis for the development of limiting tensile strain method (LTSM). Polshin and Tokar (1957) defined limiting Δ/L values of 1/3300 and 1/2500 for masonry buildings founded on sand and soft clay, respectively. Afterwards, Boscardin (1980) justified the potential reason for this difference in the proposed limits for different soils. According to Boscardin, the more tolerant Δ/L limits proposed by Polshin and Tokar for clay soils are so because building settlements occur more slowly in clay due to consolidation. Boscardin (1980) claimed that, during this slow settlement response, structural elements might be subjected to creep rather than cracking. Long afterwards, Netzel (2009) reported that ε_{crit} can be increased by 20% and 55% in case of medium and long term settlements compared to short term settlements. Similar Δ/L limits were adopted in the Russian Building Code (USSR, 1955) at almost the same time as Polshin and Tokar (1957). In Table 2.2, these values are listed for two different types of soil conditions. For sand soils, the limiting Δ/L of plain brick wall dwellings is defined as 1/3333 and 1/2000 for L/H ratios lower than 3 and higher than 5, respectively. For softer clay soils, the

limiting Δ/L of plain brick wall dwellings is defined as 1/2500 and 1/1430 for L/H ratios lower than 3 and higher than 5, respectively. According to these values, buildings with $L/H \leq 3$ (i.e., stiffer in terms of bending and more vulnerable to shear effects) will reach the cracking strain soon after having a more limited tolerance of Δ/L in comparison with the buildings with $L/H \geq 5$ (i.e., less stiff in terms of bending and less vulnerable to shear effects). In this table, for steel and RC frame structures, limiting values are defined in terms of β and multiples of L . For both steel and RC structures, the limiting β value is $1/500 L$ (no damage is expected for $\beta < 1/500 L$). Herein, L denotes the distance in metres between the foundation centres.

Table 2.2 : Limiting Δ/L and β values introduced by the USSR, 1955.

Description of standard value	Subsoil		Unit
	Sand and clay in hard conditions	Clay in plastic conditions	
Difference in the settlement of civil and industrial building column foundations:			
(a) for steel and RC frame structures	$1/500 L$	$1/500 L$	(β)
(b) for end rows of columns with brick cladding	$1/140 L$	$1/1000 L$	
(c) for structures where auxiliary strain does not arise during the non-uniform settlement of foundations	$1/200 L$	$1/200 L$	
Relative deflection of plain brick walls:			
(a) for multi-story dwellings and civil buildings			
at $L/H \leq 3$	1/3333	1/2500	(Δ/L)
at $L/H \geq 5$	1/2000	1/1430	
(L-length of the deflected part of the wall; H-height of the wall from the foundation footing)			
(b) for one-story mills	1/1000	1/1000	

In the study of Meyerhof (1956), distortion limits of frame and load-bearing wall structures were investigated. β values of 1/250 for open frames, 1/500 for infilled frames and 1/1000 for load-bearing walls were suggested as safety limit. Note that the limiting β value proposed by Meyerhof (1956) for load-bearing walls is more conservative than the value proposed by Skempton and McDonald (1956).

Grant, Christian, & Vanmarcke (1974) reported that crack initialization was observed in the walls of frame and masonry structures after a Δ/L of 1/1000.

Herein, it is useful that some conversions between β and Δ/L deformation measures were also suggested. Boscardin (1980) proposed that, for a certain damage level, the maximum β value is approximately two times Δ/L provided that the building is in a circular ground settlement profile. Afterwards, this rate was updated by Son and Cording (2005) to $\beta \cong 2 \Delta/L$ for minor cracking or elastic building conditions and $2 \Delta/L \leq \beta \leq 4 \Delta/L$ for severe cracking conditions. From this point of view, a consistency is seen between limiting Δ/L and β values proposed by Polshin and Tokar (1957) and Meyerhof (1956).

As a more up-to-date source, Eurocode 7 (CEN, 2007) can be taken into account. A general range from approximately 1/2000 to approximately 1/300 for a maximum allowable β is suggested by Eurocode 7 to preserve serviceability conditions for different types of structures, such as open frame structures, infilled frames and load-bearing brick walls. While a maximum β value of 1/500 is defined as acceptable for many structures, a β of 1/150 is evaluated as the ultimate limit value. This limit is the same as the severe distortion limit proposed by Skempton and McDonald (1956). The ultimate case is defined as unacceptable cracking or jamming of the doors in the structure. Furthermore, it is evaluated that total settlements up to 50 mm and differential settlements between adjacent columns up to 20 mm are often acceptable. For larger total and differential settlement cases, it is advised to check also if the β values are in the range of 1/2000-1/300. The validity of these values are limited with only uniformly loaded ordinary structures and only sagging-type settlement modes. Eurocode 7 (CEN, 2007) recommends that these limiting values should be halved if they are used for a hogging-type settlement mode, which is less tolerable by many structures.

2.3.2 Analytical methods : limiting tensile strain method (LTSM)

2.3.2.1 Fictitious elastic beam analogy of Burland and Wroth (1974)

Burland and Wroth (1974) extended Polshin and Tokar's (1957) study and put forward a deep elastic beam analogy for masonry (load-bearing wall structures) and frame structures (open and infilled frames) to calculate simply ε_{crit} . Burland and Wroth proposed that ε_{crit} is the most appropriate parameter to detect the onset of visible cracking. Because ε_{crit} is independent of the loading direction and deflection mode (sagging or hogging), it can be used under any loading circumstance. Herein, it has to

be highlighted that ε_{crit} is used interchangeable with the limiting tensile strain (ε_{lim}) after Burland et al. (1977) which allows to define damage classes well beyond material cracking. This is useful while defining damage class for a given Δ/L . The two dimensional (2D) fictitious deep beam represents the real structure. Although this approach yields fairly rough representations of real structures, LTSM has been widely used to assess the effect of tunnelling-induced ground settlements on structures. The fictitious deep beam is assumed to be elastic, isotropic and simply supported. Soil-structure interaction is ignored and only free-field ground settlements are considered. The model of the fictitious deep beam as considered by Burland and Wroth (1974) is given in Figure 3. The midpoint deflection (Δ) is considered to occur in either the hogging or sagging ground settlement zone. L denotes the length of the beam (in reality, it may correspond to the deflected or entire length of the structure), and H is the height of the beam. In reality, H corresponds to the building height from the foundation level up to the eaves. The roof is not included in the calculations.

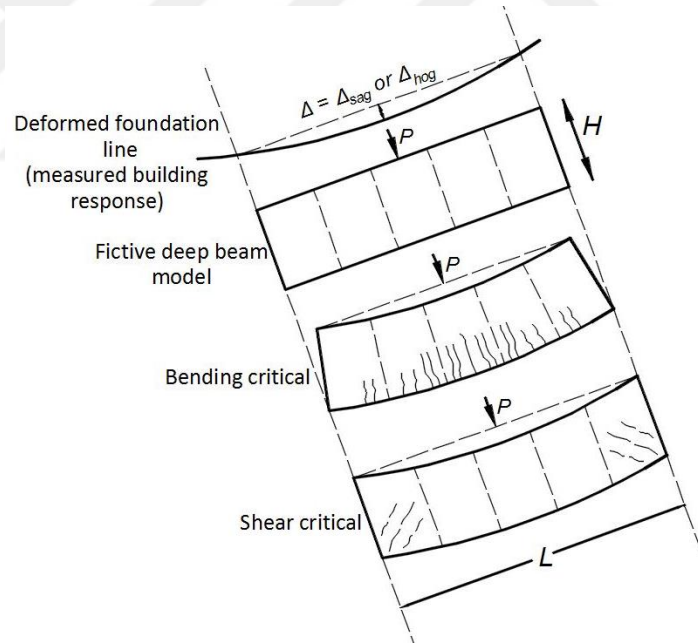


Figure 2.5 : Fictitious deep beam model (Burland and Wroth, 1974).

The problem is solved by using the beam equation given in equation 2.13. In this equation, E and G are assumed to indicate the elasticity and the shear modulus of the represented building (or a wall of a masonry building), respectively. However, they are always taken into account as a relative stiffness in the form of E/G alluding to structural longitudinal/shear stiffness ratio. I is the second moment of the area of the fictitious deep beam. P corresponds to a central point load. This equation was

previously developed by Timoshenko (1957) for central loaded and simply supported elastic beams considering the shear coefficient $k=1.5$. Note that this equation accounts for the combined contribution of bending and shear to the midpoint deflection.

$$\Delta = \frac{PL^3}{48EI} \left[1 + \frac{18EI}{L^2HG} \right] \quad (2.13)$$

Netzel (2009) proposed that the shear coefficient considered in this equation ($k=1.5$) should be updated with $k=1.2$ to prevent an overestimation of the shear deflection contribution.

If this equation is adapted to express the vertical settlement response of a building, P does not correspond to a true central load but instead refers to a factor revealing a deformation mode, as in the case of a real ground settlement. To justify the validity of the central loading in the representation of building deformation due to settlement, Burland and Wroth (1974) investigated the effect of two additional loading and deformation cases. These are a uniformly distributed loading case and a circular deflection mode. They observed that crack initialization is not sensitive to the use of these alternative loading and deflection mode applications. Therefore, it was suggested to use the central point loading P , which remarkably simplifies the calculations.

If equation 2.13 is rewritten in terms of maximum bending strain ε_{bmax} and Δ/L is isolated, Δ/L can be related to ε_{bmax} as shown in equation 2.14. This equation applies to the case in which the first visible crack is assumed to arise in bending due to sagging or hogging of the structure. ε_{bmax} denotes the maximum tensile strain that occurs at the outermost fibre of the fictitious model beam once the induced Δ/L is completely achieved. In equation 2.14, t represents the neutral axis depth from the edge of the beam in tension.

$$\frac{\Delta}{L} = \left[\frac{L}{12t} + \frac{3EI}{2tLHG} \right] \varepsilon_{bmax} \quad (2.14)$$

Similarly, equation 2.13 can be rewritten in terms of the maximum diagonal strain ε_{dmax} . If Δ/L is isolated, Δ/L can be related to ε_{dmax} as shown in equation 2.15. This equation applies to the case in which the first visible crack is assumed to arise in the diagonal direction due to sagging or hogging of the structure. ε_{dmax} denotes the

maximum tensile strain that occurs in the diagonal direction once the induced Δ/L is completely achieved.

$$\frac{\Delta}{L} = \left[1 + \frac{HL^2G}{18EI} \right] \varepsilon_{dmax} \quad (2.15)$$

Equations 2.14 and 2.15 are particularly derived to establish separate expressions between Δ/L and the maximum tensile strains (ε_{bmax} and ε_{dmax}) that might be responsible for potential tensile cracking in either bending or shear. Note that ε_{bmax} and ε_{dmax} are not necessarily critical tensile strain at this stage. To determine a damage level, the calculated maximum tensile strains (ε_{bmax} or ε_{dmax} , whichever is greater) is compared to predefined limiting tensile strain boundaries ($\varepsilon_{lim(low)}$ and $\varepsilon_{lim(up)}$) listed in Table 2.3 proposed by Burland et al. (1977) and modified by Boscardin and Cording (1989).

Table 2.3 : Damage classification table (modified from Burland et al., 1977 and Boscardin & Cording, 1989).

Category of damage	Damage class	Description of typical damage and ease of repair ^{a,b}	Approximate crack width ^c (mm)	ε_{lim} (%) After Boscardin and Cording (1989)	
				$\varepsilon_{lim(low)}$	$\varepsilon_{lim(up)}$
Aesthetic damage	0-Negligible	Hairline cracks	Up to 0.1 mm	0.000	0.050
	I-Very slight	Fine cracks that can easily be treated during normal decoration.	Up to 1 mm	0.050	0.075
	II-Slight	Cracks can be easily filled. Cracks are visible externally.	Up to 5 mm	0.075	0.150
Functional damage affecting serviceability	III-Moderate	The cracks require some opening up and can be patched by a mason.	5 to 15 mm or a number of cracks larger than 3 mm	0.150	0.300
	IV-Severe	Includes large cracks. Extensive repair work is required.	15 to 25 mm but also depends on the number of cracks	> 0.300	
Structural damage affecting stability	V-Very severe	Beams lose bearing, walls lean and require shoring, and there is a danger of structural instability.	Usually larger than 25 mm but also depends on the number of cracks	> 0.300	

Notes:

^a Location of damage in the building or structure must be considered when classifying the degree of damage.

^b Descriptions are shortened for brevity. Refer to Burland et al. (1977) for full descriptions.

^c Crack width is only one aspect of damage and should not be used alone as a direct measure of it.

2.3.2.2 Combining the maximum tensile strain with horizontal strain (Boscardin and Cording, 1989)

Ground movements induced by excavations, such as tunnelling, open-cut and mining, involve significant horizontal displacement components. Boscardin and Cording (1989) reported that, horizontal strains transferred from the ground to the structure can remarkably decrease the settlement tolerances of buildings together with bending and diagonal tensile strains. Since the fictitious deep beam approach by Burland and Wroth (1974) does not include the effect of horizontal strain, Boscardin and Cording considered this effect by combining the average horizontal strain (ε_h) with ε_{bmax} and ε_{dmax} values.

Horizontal strain is assumed to be equally distributed over the height of the structure. In reality, horizontal strains are much higher in the lower parts of structures that are close to the foundation (Son and Cording, 2005). While $S_{h,A}$ and $S_{h,B}$ are the monitored horizontal movements at A and B sections of a wall, ε_h can be calculated as shown in equation 2.16. ΔS_h denotes the differential horizontal movement (equation 2.17) and L shows the initial distance between these two sections. For structures with individual footings (frame structures), differential horizontal movement between adjacent footings is taken into account to calculate average horizontal strains. If there is no other information such as monitoring results, Boscardin and Cording (1989) suggested that ε_h value acted on a brick wall can be assumed as 0.5 to 1 times the change in the slope of the free-field ground settlement profile for braced excavations and tunnelling and 1 to 1.5 times the change in slope of the free-field ground settlement profile for excavations with cantilever walls. It has to be highlighted that this assumption can be valid as long as there is no tensile reinforcement in the footings or in the walls.

$$\varepsilon_h = \frac{\Delta S_h}{L} \quad (2.11)$$

$$\Delta S_h = S_{h,A} - S_{h,B} \quad (2.12)$$

The combined tensile strain, namely the total tensile strain (ε_t), is then calculated as shown in equations 2.18 and 2.19. While ε_{bmax} and ε_h are easily summed (equation 2.18), ε_{dmax} and ε_h are combined by using Mohr's Circle (equation 2.19). The result of equation 2.18 or equation 2.19, whichever is greater, is taken and compared with

the limiting tensile strain boundaries listed in Table 2.3 to determine the corresponding final damage class.

$$\varepsilon_t = \varepsilon_{bmax} + \varepsilon_h \quad (2.13)$$

$$\varepsilon_t = \frac{\varepsilon_h}{2} + \sqrt{\left(\frac{\varepsilon_h}{2}\right)^2 + \varepsilon_{dmax}^2} \quad (2.14)$$

If β is selected as the deformation measure instead of Δ/L (it is thought that damage is to be governed by the combined effect of shear and horizontal strain), a different combination proposed by Son and Cording (2005) can be used. According to Son and Cording, for given values of β and ε_h , total tensile strain ε_t in a part of structure can be derived in a similar manner of calculating principal tensile strain ε_p . ε_t (equal to ε_p) is calculated using equation (2.20), where θ_{max} is the crack orientation measured as the angle between the plane on which ε_t acts and the vertical plane. After comparing the calculated ε_t with the limiting tensile strains boundaries listed in Table 2.3, the damage class is determined.

$$\varepsilon_t = \varepsilon_p = \varepsilon_h \cos^2 \theta_{max} + \beta \sin \theta_{max} \cos \theta_{max} \quad (2.5)$$

$$\tan(2\theta_{max}) = \frac{\beta}{\varepsilon_h} \quad (2.6)$$

The presence of RC mat foundations or strip foundations connected by continuous slabs provides a significant tensile restraint. Consequently, horizontal ground movements are substantially reduced or inhibited at the foundation level (Boscardin & Cording, 1989; Burland, Mair, & Standing, 2004; Son & Cording, 2007). On the other hand, the presence of foundation piles in the proximity of an excavation can reduce the horizontal ground movements (Basile, 2014; Mroueh & Shahrour, 2002).

The effect of RC continuous and individual footings of frame structures was numerically investigated by Goh and Mair (2014). They found that, while horizontal strains are usually insignificant for frame structures with continuous footings, they are important for frames with individual footings even when the axial stiffness of the frame building is high. This result is in line with the findings of Laefer et al. (2009) in which scaled RC frame specimens with individual column footings were tested and

numerically simulated. Laefer et al. observed a significant increase in the drift ratio of especially the first storeys of frame structures when the horizontal ground movements are included.

2.3.2.3 Damage classification and practical design charts

As aforementioned, damage class is decided by comparing calculated maximum total tensile strain to predefined limiting tensile strain boundaries ($\varepsilon_{lim(low)}$, $\varepsilon_{lim(up)}$) presented in Table 2.3. Note that this table was priorly proposed by Burland et al. (1977) and modified after Boscardin and Cording (1989) by adding $\varepsilon_{lim(low)}$ and $\varepsilon_{lim(up)}$ boundaries. As shown, defined damage categories are essentially based on ease of damage repair. Therefore, this classification applies mostly to load-bearing masonry walls (brick & stone). However, there are cases in which it was used for infill walls of frame structures (as in the case study of the RC Elizabeth House building by Mair & Taylor, 2001). It is thought that these damage categories also require verification for different masonry wall typologies having different construction techniques (i.e., multi-leaf walls). Burland et al. (1977) noted that crack widths in this classification are to be used as an additional indicator rather than being a direct measure of damage. They also highlighted that damage is usually difficult to quantify because it depends on many subjective criteria. On the other hand, damage that is acceptable in a region or for one type of building might not be acceptable for other cases. Therefore, although this damage classification is widely used, users should keep in mind that it reflects generalized cases.

For the practical use of the LTSM, some diagrams based on the damage classifications given in Table 2.3 were proposed by Boscardin and Cording (1989) and Burland (1997) (Figure 2.6). As shown in Figure 2.6 (a), the diagram proposed by Boscardin and Cording (1989) shows the limiting curves that are boundaries of different damage classes predefined in Table 2.3. To obtain these curves, fictitious deep beam (with $L/H = 1$ and $E/G = 2.6$) equations derived for hogging ($t = H$) are used together with the given limiting tensile strain boundaries $\varepsilon_{lim(low)}$ and $\varepsilon_{lim(up)}$. In order to obtain presented β values in this diagram (β_{max} at the support sections of centrally loaded deformed fictitious deep beam), equation 2.22 is used. For typical values of L/H and E/G , this expression gives β_{max} values that vary between 2 or 2.3 times Δ/L .

$$\beta_{max} = \frac{3\Delta}{L} \left[\frac{1 + 4 \left(\frac{E}{G} \right) \left(\frac{H^2}{L^2} \right)}{1 + 6 \left(\frac{E}{G} \right) \left(\frac{H^2}{L^2} \right)} \right] \quad (2.7)$$

A limited amount of case study data are also provided to validate the damage categories. The case study data consists of infilled steel frame, infilled wood frame and brick masonry structures that are mostly small to medium in size. Boscardin and Cording reported that none of these structures was capable of modifying the free-field ground movements due to their relatively small/medium size and absence of a grade beam.

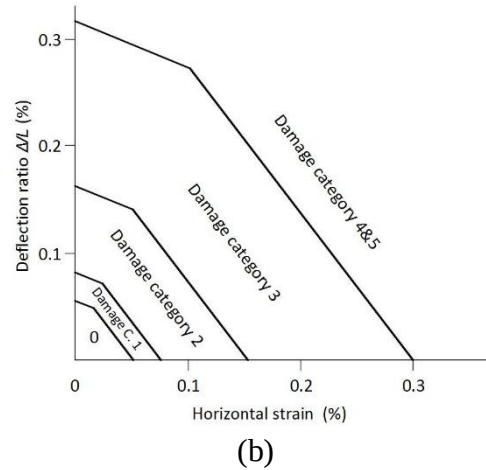
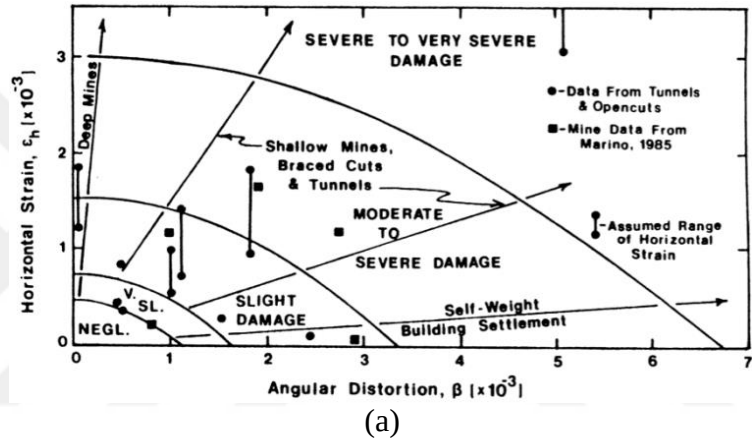


Figure 2.6 : Damage prediction diagrams of (a) Boscardin and Cording (1989) and (b) Burland (1997).

The other diagram proposed by Burland provides a similar interaction between ϵ_h and Δ/L (Figure 2.6 (b)). By combining equations 2.14 & 2.15 with 2.18 & 2.19 respectively, and setting $\epsilon_{bmax} = \epsilon_{lim}$ or $\epsilon_{dmax} = \epsilon_{lim}$, the curves given in Figure 2.6 (b) are obtained for each damage class given in Table 2.3. Note that for the derivation

of these curves the hogging condition is considered ($t = H$) along with $L/H = 1$ and $E/G = 2.6$. As seen, for a given Δ/L , as the acted horizontal strain increases, the damage class shifts from lower to higher damage. Burland criticises Boscardin and Cording's diagram as to that maximum bending strains ε_{bmax} are ignored. This criticism can be seen as an extension of another discussion where the fact that the use of beta is mostly being related to shear behaviour of the structure was asserted. In addition, Burland states that equation 2.22 used by Boscardin and Cording to calculate β_{max} from Δ/L is very sensitive to the load distribution.

2.3.2.4 Laminate beam approach (Finno et al., 2005)

Finno et al. (2005) proposed a laminate beam approach to replace the simplistic stiffness ratio definition (E/G) implemented in LTSM. They reported that fictitious deep beam approach is not sufficient to describe the full range of responses of buildings subjected to ground movements because it does not account for the variations in structural systems. According to Finno et al. (2005), defining accurate values for E/G and the neutral axis position for a complicated multi-story building is difficult. They developed a laminate (sandwich) beam model as shown in Figure 2.7. In this model, structures are assumed as laminated beams, which are comprised of different layers. Each layer corresponds to either vertical load-bearing elements (masonry walls or RC columns) or slabs of a multi-story building. The slabs are assumed to resist bending effects due to vertical ground movements. The walls (load-bearing or infill walls) and columns are assumed to resist shear effects due to vertical ground movements. The height of each story (y_i) can be considered separately. Indices i show floor number. The location of the neutral axis of the entire laminate beam (λH) can be calculated considering the distance of each layer from the bottom of the laminate beam (h_i) and the cross-section area of the layers contributing to the bending resistance (A_i). Then, the second moment of the area of the laminate beam (I) is calculated according to the position of this neutral axis.

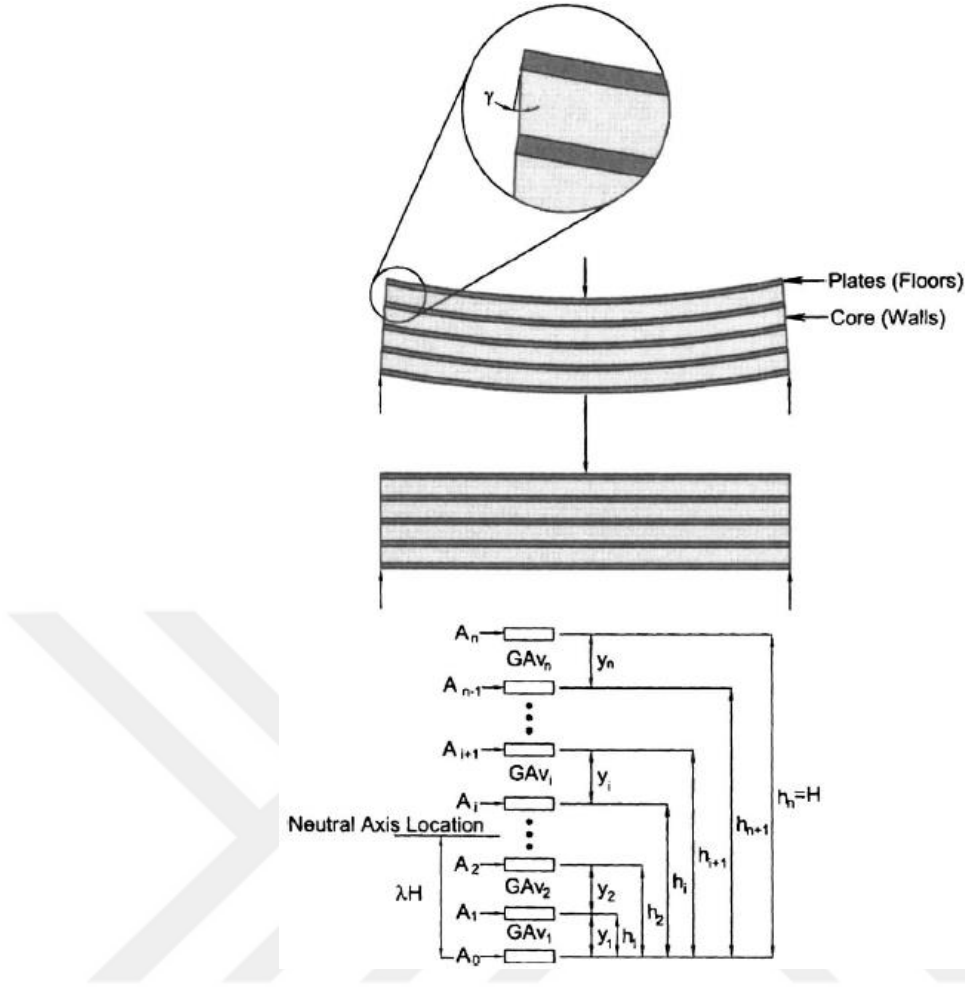


Figure 2.7 : Laminated beam model (Finno et al., 2005).

They proposed the use of a new stiffness ratio EI/GA_v instead of E/G to account for the variations in bending and shear stiffness of a structure. In this expression, A_v is the shear-resisting cross-section area of each layer. The limiting values of Δ/L for each story can be calculated for given critical values of shear strain (γ_{crit_i}) using equation 2.23. On the other hand, for a given (monitored) Δ/L value induced shear strain can also be found by this equation. In this equation, V_i is the shear force acting each floor, V is the total shear force acting on laminated beam, $(GA_v)_{eqv}$ is the equivalent shear stiffness of the laminated beam and γ_{add} is the additional shear strain. Remember that additional shear strain γ_{add} is involved to consider complex settlement profile modes, such as in Figure 2.1 (a-ii). As seen, the presence of γ_{add} reduces tolerable Δ/L value.

$$\frac{\Delta}{L} = \left[\frac{L^2(GA_v)_i}{24EI(V_i/V)} + \frac{(GA_v)_i}{2(V_i/V)(GA_v)_{eqv}} \right] (\gamma_{crit_i} - \gamma_{add}) \quad (2.23)$$

The case history data of a school building with infilled and masonry brick walls was

used to validate the proposed approach. It was reported that the monitored cracks on the infill walls at different story levels could be predicted reasonably by this approach, but they could not be predicted by fictitious deep beam approach due to its oversimplification of structural details and exclusion of additional shear strain.

2.3.2.5 Relative stiffness method (Potts and Addenbrooke, 1997)

In the LTSM, the interaction between soil and structure is neglected and it is assumed that free-field ground displacements are fully transferred from soil to structure without any modification. Whereas, in reality, surface structures usually modify and limit the tunnelling- and excavation-induced ground movements. One of the pioneering studies in which the dual interaction between the soil and structure during an ongoing tunnelling excavation is investigated is that of Potts and Addenbrooke (1997). Burland et al. (2001) nominated this study as a promising development. The relative stiffness method is originally based on the results of parametric finite element analyses in 2D. In practice, design curves derived from these analyses are used. Potts and Addenbrooke's analyses included equivalent simple surface beams representing masonry and frame structures with different features, such as different numbers of stories (5, 3, 1 and for a single slab), building length (perpendicular to tunnel line) (B) and eccentricities (e) are modelled. Eccentricity (e) denotes the offset of the 'beam' centreline with respect to the tunnel centre line, as shown in Figure 2.1 (a). An equivalent bending stiffness $(EI)_{eqv}$ value is calculated based on each floor's bending stiffness according to parallel axis theorem. Also, an equivalent axial stiffness $(EA)_{eqv}$ is calculated by summing the axial stiffness of slabs at each floor. Potts and Addenbrooke performed their analyses using representative surface beam models with $(EI)_{eqv}$ and $(EA)_{eqv}$ stiffness. They also considered coupled soil-structure interaction. Relative bending (ρ^*) (related to flexural deflection response) and relative axial stiffness (α^*) (related to horizontal deformation response) concepts were introduced for the first time to account for the stiffness of both the soil and structure together. ρ^* and α^* are calculated by using equation 2.24. In this equation, while E_s is the soil stiffness, H is taken as half length of the represented structure ($B/2$).

$$\rho^* = \frac{(EI)_{eqv}}{E_s H^4} (m^{-1}); \alpha^* = \frac{(EA)_{eqv}}{E_s H} (\text{unit value}) \quad (2.24)$$

Using very similar tunnelling scenarios (1.5% volume loss), finite element analyses with surface beams and without surface beams (free-field) were conducted. To quantify the magnitude of modification, modification factors based on the deflection ratios ($M^{(\Delta/L)_{sag}}$ and $M^{(\Delta/L)_{hog}}$) and horizontal strains ($M^{\varepsilon_{hc}}$, $M^{\varepsilon_{ht}}$) were proposed. Modification factors are calculated through equations 2.25 and 2.26. While subscripts ‘sag’ and ‘hog’ denote sagging and hogging, superscript ‘f’ refers to free-field values. To identify whether modification factors for horizontal strains are derived in compression (for sagging) or tension (for hogging), the indices ‘c’ and ‘t,’ which are the abbreviations of compression and tension, are used. Note that while a modification factor of ‘one’ corresponds to a fully flexible response, a modification factor approaching ‘zero’ indicates very rigid response.

$$M^{(\Delta/L)_{sag}} = \frac{(\Delta/L)_{sag}}{(\Delta/L)_{sag}^f}; M^{(\Delta/L)_{hog}} = \frac{(\Delta/L)_{hog}}{(\Delta/L)_{hog}^f} \quad (2.25)$$

$$M^{\varepsilon_{hc}} = \frac{\varepsilon_{hc}}{\varepsilon_{hc}^f}; M^{\varepsilon_{ht}} = \frac{\varepsilon_{ht}}{\varepsilon_{ht}^f} \quad (2.26)$$

When the interaction analyses were repeated for the most likely and realistic range of axial stiffness values ($\alpha^* > 0.5$), it was observed that there were certain trends between ρ^* and the modification factors depending on the eccentricity ratio e/B (Figure 2.8). As shown in Figure 2.8, both $M^{(\Delta/L)_{sag}}$ and $M^{(\Delta/L)_{hog}}$ tend to decrease as ρ^* increases for all e/B ratios. Both $M^{(\Delta/L)_{sag}}$ and $M^{(\Delta/L)_{hog}}$ have small values for $\rho^* > 10^{-2} m^{-1}$. This proves that increasing the bending stiffness (ρ^*) results in a substantial decrease in the building distortion. As expected, as the building moves away from the tunnel centre line (e/B increases), $M^{(\Delta/L)_{sag}}$ decreases, but $M^{(\Delta/L)_{hog}}$ increases.

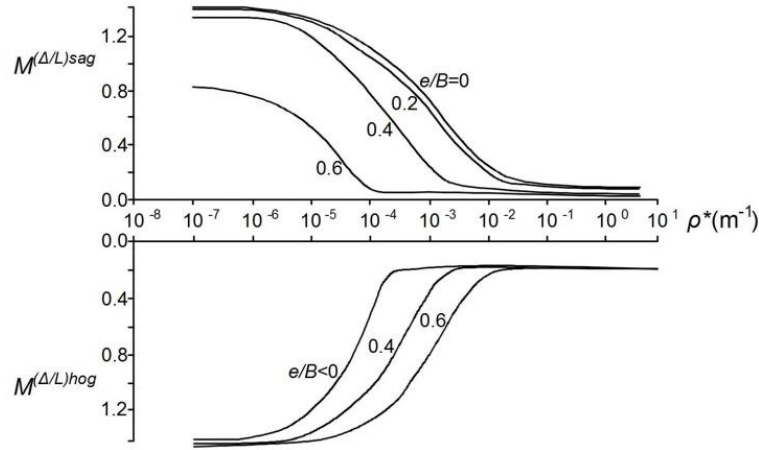


Figure 2.8 : Design curves for modification factors of the deflection ratio (modified from Potts and Addenbrooke, 1997).

When a similar procedure was conducted for the most likely and realistic range of bending stiffness ($5.18 \times 10^{-8} < \rho^* < 5.18 \times 10^{-1} \text{ m}^{-1}$), it was observed that both $M^{\varepsilon_{hc}}$ and $M^{\varepsilon_{ht}}$ decreased with increasing α^* . As seen in Figure 2.9, for $\alpha^* \geq 10$, transferred horizontal strains substantially diminish. Similarly, as the building moves away from the tunnel centre line (e/B increases), $M^{\varepsilon_{hc}}$ decreases, but $M^{\varepsilon_{ht}}$ increases. It should be made clear that these curves given in Figures 2.8 and 2.9 show the upper bound of obtained modification factors.

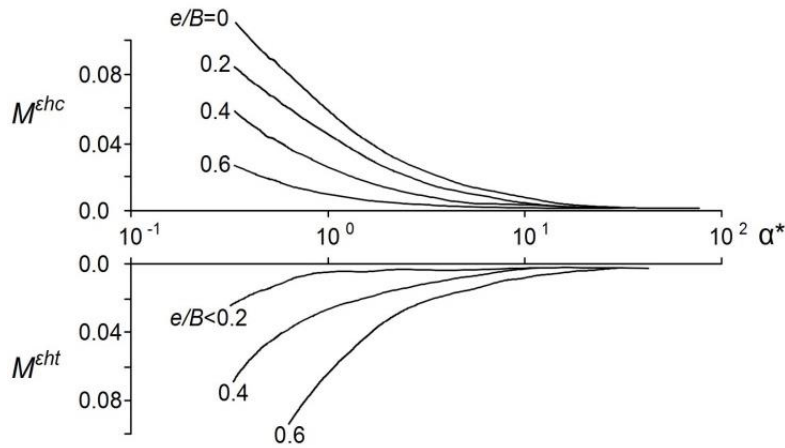


Figure 2.9 : Design curves for modification factors of horizontal strain (modified from Potts and Addenbrooke, 1997).

For known values of ρ^* , α^* and e/B , modified deflection ratios and modified horizontal strains are calculated using equations 2.27 and 2.28, respectively. The free-field deflection ratio and horizontal strain values in these equations can be taken as predicted by the Gaussian Normal Distribution Curve Method (Peck, 1969).

$$\begin{aligned}
(\Delta/L)_{sag} &= M^{(\Delta/L)_{sag}}. (\Delta/L)^f_{sag} \\
(\Delta/L)_{hog} &= M^{(\Delta/L)_{hog}}. (\Delta/L)^f_{hog}
\end{aligned}
\tag{2.27}$$

$$\varepsilon_{hc} = M^{\varepsilon_{hc}}. \varepsilon^f_{hc} ; \varepsilon_{ht} = M^{\varepsilon_{ht}}. \varepsilon^f_{ht}
\tag{2.28}$$

In the last step, binaries of $[(\Delta/L)_{sag}, \varepsilon_{hc}]$ or $[(\Delta/L)_{hog}, \varepsilon_{ht}]$ can then be input to determine the damage class according to Burland's diagram (Figure 2.6 (b)). One of the most important outcomes of this study is the demonstration of the increase of building vulnerability with increasing building length perpendicular to tunnel line and decrease of building vulnerability with increasing building height. This conclusion is in line with the findings reported by Goh and Mair (2014).

The relative stiffness method was subsequently improved by Franzius et al. (2006) by modifying ρ^* and α^* expressions. Additional tunnel depths were considered. Moreover, they included the building self-weight and building dimension parallel to the tunnel line (3D effects), adding a smooth soil-structure interface model.

Giardina et al. (2015b) showed that the inclusion of building self-weight in their numerical analyses increased the calculated modification factors with respect to ones calculated by equations 2.25 and 2.26. Therefore, they supported inclusion of the effect of building self-weight in the calculation of the modification factors to avoid underestimation of building distortions. Likewise, Camós et al. (2014) reported that predicted damage using the original relative stiffness method underestimated the real damages observed on the façade walls of single-story masonry buildings situated above the L9 metro tunnel excavation in Barcelona. Dimmock and Mair (2008) proposed simple modifications to reduce the estimated relative bending stiffness (ρ^*) to overcome the underestimation of actual building distortions with original relative stiffness method, particularly in hogging. According to Dimmock and Mair, for masonry buildings in hogging, the contribution of the walls, which are working mostly in tension, is very little or zero. Thus, they suggested considering only the foundation slab in the calculations of relative bending stiffness. Additionally, they proposed to reduce the estimated relative bending stiffness ten times to consider the potential effect of wall openings. The suggested modifications depend on case study observations.

2.3.3 Numerical studies of soil-structure interaction in the presence of tunnelling- and excavation-induced ground movements

Numerical techniques enabled multi-parameter analyses in which geometrical and mechanical properties can be considered more realistically. Furthermore, the addition of tunnel construction and/or soil models to the finite element analyses allowed the investigation of coupled tunnel-soil-building or coupled soil-building behaviour.

Amorosi et al. (2014) performed 2D coupled (soil-structure interaction is considered in a unique finite element analysis) and uncoupled (free-field settlements are directly imposed to structure base in a separate analysis without considering soil-structure interaction) analyses to investigate the settlement response of an ancient masonry wall structure during tunnelling. They used nonlinear anisotropic and elasto-plastic material models for masonry and soil, respectively. Use of nonlinear inelastic material models (having cracking capability) for masonry provided a better insight of real structural behaviour. The authors observed a reduction of plastic strains in masonry wall and of horizontal displacements in soil when soil-structure coupled analysis is conducted. Furthermore, coupled analysis results were compared with the monitored data, and good agreement was observed.

Son and Cording (2005) and Giardina et al. (2013) also highlighted that, misleading results are more likely if the analyses relying on only elastic material properties are used in the cases cracking strongly governs the building settlements. Son and Cording (2005) investigated the response of elastic and inelastic building walls to excavation-induced ground movements. They showed that, as the relative soil/structure stiffness increases (or building stiffness decreases), building distortions increase. In this case, they observed that the building moves closer to the free-field ground settlement profile and that rigid rotations decrease. On the other hand, if the soil/structure stiffness ratio decreases (or the building stiffness increases), the building distortions are reduced. Moreover, the ground settlement profile is flattened, and rigid rotations increase. A similar tendency was observed for walls with increased brick-mortar tensile strength as well. These findings are also in line with the relative stiffness definitions of Potts and Addenbrooke (1997). Giardina et al. (2013) reproduced a masonry façade experiment conditions using 2D semi-coupled (soil-structure interaction is simulated by interface elements assigned to the base of the structure) numerical analyses.

Giardina et al. found that final damage is significantly affected by inelastic material properties of masonry and the normal stiffness value of the assigned interface.

Settlement response of different structure types, namely load-bearing masonry and infilled frame, to tunnelling-induced ground movements was investigated by Son (2016). The author used distinct element method (DEM) in which brick units and frame elements (beam and column) are elastic, yet brick-mortar joints are inelastic. He considered soil-structure interaction by a finite soil and interface layer. The author expressed that, structural response is less dependent on the structure and soil properties if the magnitude of ground settlement is low. When ground settlement becomes sufficiently large, the effect of soil property decreases but the effect of structure type increases.

Mroueh and Shahrour (2003) and Fagnoli et al. (2015) conducted 3D finite element analyses and investigated the effect of soil-structure interaction during tunnelling. They both considered RC frame buildings. Mroueh and Shahrour (2003) observed that internal forces of the frame elements are remarkably overestimated in uncoupled analysis. This result highlights how extremely conservative evaluations can be expected if ground settlement modification caused by structure's stiffness is ignored. Additionally, Mroueh and Shahrour (2003) and Fagnoli et al. (2015) noted remarkable underestimation in the internal forces when building self-weight is ignored. According to Mroueh and Shahrour (2003), the reason of this underestimation is a ground settlement reduction due to decreasing plastic deformations around column footings. Indeed, significant stress concentrations were observed when the building self-weight is considered. Furthermore, they noted that differential settlements between adjacent frames also decrease if the building self-weight is neglected. These findings were later supported by the experimental results of Laefer et al. (2009). Fagnoli et al. (2015) used three different modelling techniques, from simple to advanced, to reveal the effect of model complexity. The simplest building model was comprised of an equivalent plate. The second model consisted of only the building portion, which was buried in the soil. In this model, the weight of the upper portion of the building acted on the buried portion (mainly on the columns). In the detailed model (third), external infill walls were individually modelled utilizing weightless cross-bracings placed in building frames. The results of the second model showed good agreement with the monitored data and the detailed model despite the model simplicity. The equivalent

plate approach overestimated the structure stiffness, yielding highly inaccurate displacement results in comparison with the monitored data. According to Fagnoli et al. (2015), success of the second model proved that the stiffness of the upper portions of the structure did not contribute much to the settlement response of this building. Although this result seems contradictory to the previous findings, it has to be noted that this conclusion might be because there were three RC strip ($w \times d \times l = 1.30 \times 0.65 \times 3.0$ m) and three raft ($w \times d \times l = 3.10 \times 0.70 \times 1.90$ m) foundations, which might have the potential to govern the building settlement response. Furthermore, Losacco et al. (2014) noted that such a simplification cannot be effective because it ignores significant torsional effects can arise due to tunnel progression along a line that is skewed (not exactly parallel or perpendicular) with respect to the building.

Giardina (2013) simulated tunnelling advance in 3D. The author reported that neglecting developing longitudinal settlements during a tunnelling operation can cause underestimation of building damage depending on volume loss and building position in the settlement trough.

The literature survey reveals that there is a trade-off with respect to the specification and the variety of the parameters in numerical modelling of soil-structure interaction. The importance of consideration of soil-structure interaction is emphasized. In general, although 3D modelling was found to be more realistic, 2D studies were frequently preferred for multi-parameter analyses. Semi-coupled analyses to be performed in 3D can be advantageous to consider soil-structure interaction realistically. The use of this technique can minimize the computational burden.

2.3.4 LTSM amidst other methods

To determine the ultimate risk level of the buildings subjected to tunnelling-induced ground movements, a three-staged process is conducted: preliminary assessment, second-stage assessment and detailed assessment (Mair et al., 1996). LTSM is a widely accepted practical tool for second-stage assessment of, in particular, brick masonry walls affected by tunnelling-induced ground movements. Because the final risk decision for the majority of investigated buildings is to be made in the second-stage assessment, the accuracy of the LTSM is important. If the accuracy and precision of the LTSM is increased, it not only enhances the accuracy of final risk decisions but

also may reduce the number of buildings that are to be assessed by expensive and time-consuming methods such as finite element methods in the detailed assessment stage.

However, due to its simplistic nature, use of the LTSM cannot be sufficient. The most frequently reported deficiencies of the LTSM are presented below. These shortcomings have stimulated other methods and approaches.

- Structural differences and variety (at both the structural and material levels) cannot be sufficiently reflected. Simply using different fictitious values for E/G does not guarantee that different structures with different structural systems, compositions, materials, irregularities, etc., are represented well. When the detailed stress, strain and damage distributions are important, as in the case of frame structures, LTSM and conventional deformation measures cannot be sufficient (Laefer et al., 2009). In these cases, finite element analyses can be crucial.
- The effect of different types of foundations cannot be considered. Assumptions made for the position of the neutral axis might not reflect actual cases if the structural system is complicated.
- Only one type of settlement deflection mode is considered, and the validity of this deflection mode (a deflected shape of a simply supported beam under a central load) is limited by the building length L . If the building length is too long or short, the simply supported beam approach becomes invalid (Boone, 1996; Finno et al. 2005).
- In calculations, free-field ground movements are used. Moreover, these are directly imposed on the fictitious model beam assuming there is no relative movement between building and soil. The soil-structure interaction is ignored. This assumption generally results in more conservative estimates of the building response. However, in reality, there is a dual interaction between soil and structures (Boscardin & Cording, 1989; Murphy, Gaynor and Laefer, 2010; Potts & Addenbrooke, 1997).
- Although recent works (Clarke & Laefer, 2014) have started suggesting an index-based approach to incorporate the existing condition of buildings into building risk assessment framework, initial damage to the buildings (existing cracks, etc.) is not originally accounted for in the LTSM. This may lead to

overestimation of the initial stiffness. Furthermore, widening of the existing cracks and damage accumulation during ground settlement cannot be considered. This situation might lead to un-conservative damage predictions. Fok, Neo, Goh and Wen (2012) suggested that detailed assessments should always be conducted for buildings that are found to be in poor conditions in preliminary assessment.

- Nonlinearity of building materials and stiffness degradation during settlements cannot be considered. On the contrary, the finite element analyses and laboratory tests performed by Son and Cording (2005), Laefer et al. (2009) and Giardina et al. (2012) showed that consideration of nonlinear material properties (cracking) has a significant influence on masonry wall and frame stiffness and thus overall response.
- The effect of tunnelling activities parallel to the tunnel axis is ignored. Therefore, the potential torsional response of buildings underground movements cannot be considered. Furthermore, because a plane stress condition is considered, the contributions of the other walls and slabs that are orthogonal to the considered plane are not taken into account (Giardina, 2013).
- Building self-weight is ignored. On the contrary, there are several reported results in the literature that highlight the effect of building self-weight (Boonpichetvong M, Netzel H, & Rots, 2006; Burd, Houlsby, Augarde, & Liu, 2000; Fagnoli, Gragnano, Boldini, & Amorosi, 2015; Franzius, Potts, & Burland, 2006; Giardina, DeJong, & Mair, 2015a; Mroueh & Shahrour, 2003; Laefer et al., 2009). In general, it is understood that the conformity to the ground settlement profile is increased by increased building self-weight. Thus, larger distortions and internal forces arise. In addition, redistribution of the building self-weight during on-going excavations can result in a partial embedment of the structure into the soil (Farrell, Mair, Sciotti & Pigorini, 2014).

The LTSM as a practical tool is useful in the preliminary assessment stages of the buildings as long as its limitations are considered. However, the literature review showed that the LTSM might be substantially deficient to predict real building damages due to its simplicity. Although frequently used in practice, the number of

studies in which the LTSM is evaluated / verified using the real site data is very limited. In addition, the number of projects where the project information is recorded in detail in a way that allows an in-depth analysis of the LTSM is very few.

The current thesis study aims to review the LTSM and to verify the accuracy of the estimated building damage classes for a limited number of cases using the results of the back calculations. For this purpose, building settlement data is used from real and simulated cases (Chapter 3). A two-stage numerical analysis approach based on a case study with several project uncertainties was then proposed. With this staging approach, some of these uncertainties are put-off and the building model is initially calibrated. Then, parametric analyses are performed to investigate the effect of each variable.





3. USING TRANSFER FUNCTIONS TO REVIEW THE LIMITING TENSILE STRAIN METHOD (LTSM)

3.1 Viewing the LTSM as Transfer Functions

In sub-section 2.3.2, the basic idea behind the LTSM is presented. In this section, equations 2.14 and 2.15 based on the fictitious deep beam analogy are presented as transfer functions. Thus, a new perspective is provided for easier and more independent interpretation of the relationships between deflection ratio (Δ/L), structure's geometry (L/H), stiffness ratio (E/G) and tensile strains. Setting the ε_{bmax} and ε_{dmax} values in equations 2.14 and 2.15 as ε_{crit} , these equations can be reformulated as transfer functions such as $\Delta/L = C\varepsilon_{crit}$. Herein, C corresponds to coefficients in equations 2.14 and 2.15 that are dependent on the geometrical and stiffness properties L, H, E, G, I and t .

Table 3.1 : The equations extracted from fictitious deep beam equations (equations 2.14 and 2.15) to calculate C_b and C_d coefficients.

C coefficients	Sagging ($t = H/2$)	Hogging ($t = H$)
C_b (extracted from equation 2.14 for bending critical cases)	$0.17 \frac{L}{H} + 0.25 \frac{H}{L} \frac{E}{G}$	$0.083 \frac{L}{H} + 0.5 \frac{H}{L} \frac{E}{G}$
C_d (extracted from equation 2.15 for diagonal tension critical cases)	$1 + 0.67 \left(\frac{L}{H}\right)^2 \frac{G}{E}$	$1 + 0.17 \left(\frac{L}{H}\right)^2 \frac{G}{E}$

Table 3.1 shows extracted C coefficients calculated for sagging and hogging. While C_b is for the bending critical case, C_d is for the shear critical case. For sagging buildings, the neutral axis is assumed at the middle of the fictitious deep beam (Burland and Wroth, 1974). Hence, the neutral axis depth becomes $t = H/2$ in the calculations. The second moment of the area is calculated as $I = 1 \times H^3/12$ assuming a unit width of the beam. For buildings in hogging or if significant restraints (such as RC mat foundations) exist, the neutral axis is assumed to be along the bottom edge of the fictitious beam (Burland & Wroth, 1974). In this case, the neutral axis depth shifts

from $t = H/2$ to $t = H$. I is calculated according to the new location of the neutral axis as $H^3/3$. The selected positions of neutral axis in sagging and hogging depend on the empirical observations made by Burland and Wroth. According to the authors, due to the presence of compression strains which are transferred from soil to the lower parts of the buildings in sagging, it can be assumed that the neutral axis positions at the centre line of the ‘beam’. On the other hand, the buildings with especially shallow weak foundations are largely influenced by the tensile strains transferred from the soil in a hogging zone. Burland and Wroth assumed that neutral axis should be at the bottom edge of the ‘beam’ to represent the vulnerability of these buildings. In other words, it is assumed that building walls work for tension. Boscardin and Cording (1989), Mair et al. (1996) and Son and Cording (2007) also support this assumption. The results of the numerical study by Son and Cording (2007) in which the response of a masonry model wall situated in the hogging zone is examined are in good agreement with the analytical results obtained using a fictitious cantilever deep beam model with the neutral axis along the bottom edge. According to Netzel (2009), if horizontal strains are not considered in damage calculation, the assumption of neutral axis is at the bottom edge leads to lower strains (compared to the case of neutral axis is at the centre line) in L/H range between 0.75 and 2.5. The author states that this situation is not in line with the main intention that was to increase the susceptibility of hogging buildings under only vertical ground movements.

3.1.1 The effect of the E/G ratio in LTSM shown by transfer functions

Burland and Wroth (1974) used three different E/G ratios (0.5, 2.6 and 12.5) to identify different structure types (masonry and frame structures) and to consider unique features of masonry structures such as window or door openings in load-bearing walls. According to elasticity theory, E/G equals $2(1 + \nu)$ for isotropic materials. For $\nu = 0.3$, E/G becomes 2.6. Burland and Wroth (1974) proposed $E/G = 2.6$ for massive load bearing walls (excluding openings) even if the massive masonry is not an isotropic and linear elastic material. They proposed $E/G = 12.5$ for buildings that are more flexible, such as RC open frames, so that they can conform to the settled ground profile more easily than infilled RC frames and rigid masonry wall structures. The aim of selecting a higher E/G value for these flexible structures was to consider their decreasing vulnerability in bending. For buildings with extremely high stiffness in shear (or little longitudinal stiffness), such as wall structures made of precast

concrete units, $E/G = 0.5$ was proposed for hogging. Note that proposed values were supported by only experiences of the authors without any other evidence. Furthermore these generalized values can significantly change from one building to another. Potts and Addenbrooke (1997) and Finno et al. (2005) proposed more realistic techniques to calculate the overall equivalent relative stiffness of masonry load-bearing and frame structures.

Many other researchers also empirically investigated the most reasonable values of E/G in different cases. Cook (1994) made back-calculations by modelling real masonry façade walls as fictitious deep beams. The author reported that E/G was taken as 30 in fictitious deep beam calculations to provide similar distortions with the actual façade walls. This value is remarkably higher than the values proposed by Burland and Wroth (1974). This was probably because the façade walls examined by Cook experienced mostly shear-related damages.

Son and Cording (2007) performed numerical tests to find equivalent normal and shear stiffness values for modelling masonry buildings subjected to settlement effects. They also investigated the effect of different opening ratios and brick-mortar interface properties on these equivalent stiffness values. Their findings supported the assumption that an increase in the opening ratio and/or a decrease of the shear/normal stiffness ratio of the brick-mortar interface results in dramatic increases of E/G . For a typical opening ratio and a typical range of the shear/normal stiffness ratio, an equivalent E/G of 12 was proposed.

Giardina, Hendriks, & Rots (2015b) performed sensitivity analysis using finite element modelling. They investigated the effects of many factors, including the opening ratio (0, 10 and 30%) and material parameters (elasticity modulus, tensile strength and fracture energy). After an extrapolation, they showed that E/G ratios of 2.6, 8 and 11 in the LTSM correspond to opening ratios of 0, 10 and 30% in their finite element model of masonry walls, respectively.

To visualize the effect of E/G , C coefficient versus L/H relationships can be used. The composite curves in Figures 3.1 (a) and (b) show the minimum (and thus decisive) values of C_b and C_d as a function of L/H . Each curve is obtained for a different E/G value (12.5, 2.6, 4.0 and 0.5). While the composite curves given in Figure 3.1 (a) are

derived for sagging ($t = H/2$), those given in Figure 3.1 (b) are derived for hogging ($t = H$).

Recalling the formulation of the transfer function, an increase in C coefficients results in a decrease in ε_{crit} values for a given Δ/L . Although ε_{crit} was introduced as a certain material property associated with the onset of cracking, herein, reduction in ε_{crit} corresponds to a decrease in cracking vulnerability. As shown in Figures 3.1 (a) and (b), cracking is governed by shear for a lower range of L/H values for all values of E/G . As mentioned before, L denotes the affected length of the structure. It is not always necessarily the entire length of the structure. In the initial stages of a tunnelling or excavation, buildings act as if they have a low L/H . Boscardin and Cording (1989) and Son and Cording (2007) explained this fact by highlighting the propagating nature of tunnelling- and excavation-induced ground movements. According to them, ground movements usually proceed over time, and L , impinged by the ground movements, increases progressively. The figure thus illustrates that, in the initial stages of a tunnelling or excavation (for lower values of L/H), diagonal cracks (as an indicator of the shear dominating response) in the walls are more likely to occur.

Another interesting point that was underlined by Burland and Wroth (1974) concerns the range of L/H that induces shear in sagging and hogging. As shown, for buildings in hogging, this range is twice the corresponding range for sagging for all E/G ratios. The dominating coefficient is altered from shear to bending at different values of L/H depending on the value of E/G . As the longitudinal stiffness is increased (or the shear stiffness is decreased), the range of L/H with dominating shear damage is extended in both sagging and hogging (Figures 3.1 (a) and 3.1 (b)). This result is in line with the notion that, if E/G increases, the range of L/H where shear is responsible for crack initialization is extended. For buildings with a low E/G of 0.5, almost the entire range of L/H is dominated by bending in both sagging and hogging (Figures 3.1 (a) and (b)).

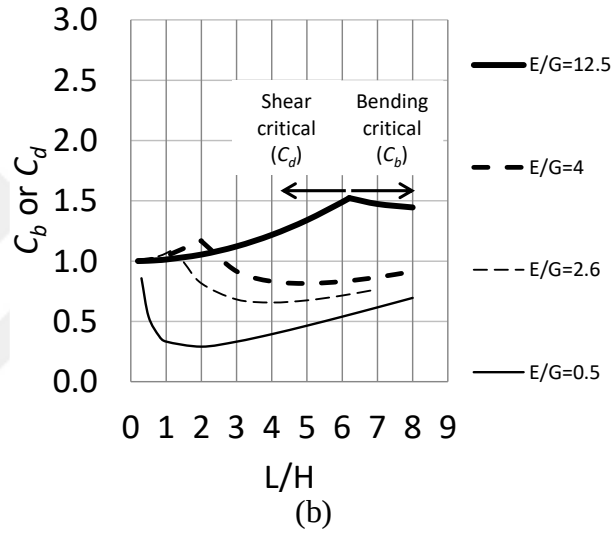
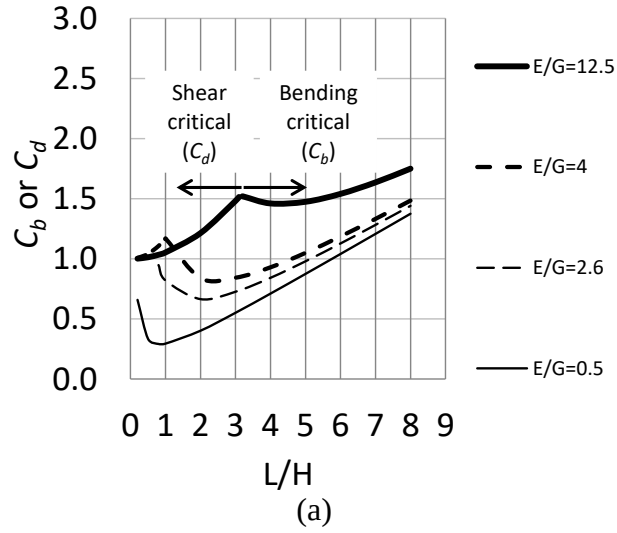


Figure 3.1 : $C - L/H$ composite curves obtained for different values of E/G : (a) for sagging and (b) for hogging.

3.1.2 Implementation of horizontal strains in the transfer function of LTSM

The effect of horizontal strains can be introduced to the transfer function concept for the hogging cases. To achieve this, transfer function equation is re-expressed in terms of the total tensile strain ε_t and the horizontal strain ε_h , as shown in equations 3.1 and 3.2. For this purpose, ε_{bmax} and ε_{dmax} are isolated in equations 2.18 and 2.19 and then expressed as $(\varepsilon_t - \varepsilon_h)$ and $(\varepsilon_t - \varepsilon_h \cos 45^\circ)$, respectively. During the isolation of ε_{dmax} from equation 2.19, the diagonal component of ε_h can be simply taken as $\varepsilon_h \cos 45^\circ$ assuming that it is in the 45° inclined plane with respect to the horizontal direction. If each side of equations 3.1 and 3.2 is linearized in ε_t , the expressions can be rewritten as shown in equations 3.3 and 3.4. Herein, the adapted coefficients C_b^* and C_d^* are functions of C_b , C_d and the $\varepsilon_h/\varepsilon_t$ ratio.

$$\Delta/L = C_b \varepsilon_{bmax} = C_b (\varepsilon_t - \varepsilon_h) \quad (3.8)$$

$$\Delta/L = C_d \varepsilon_{dmax} = C_d \sqrt{\varepsilon_t^2 - \varepsilon_t \varepsilon_h} \cong C_d (\varepsilon_t - \varepsilon_h \cos 45) \quad (3.9)$$

$$\Delta/L = \left[C_b \left(1 - \frac{\varepsilon_h}{\varepsilon_t (= 0.0006)} \right) \right] \varepsilon_t = [C_b^*] \varepsilon_t \quad (3.10)$$

$$\Delta/L = \left[C_d \left(1 - \frac{\varepsilon_h}{\varepsilon_t (= 0.0006)} \cos 45 \right) \right] \varepsilon_t = [C_d^*] \varepsilon_t \quad (3.11)$$

Total tensile strain ε_t in $\varepsilon_h/\varepsilon_t$ can be assumed as critical tensile strain of brick masonry wall at the onset of visible cracking. Herein, ε_{crit} is taken as 0.06% complying with the suggestion of Burhouse (1969). Thus $\varepsilon_h/0.0006$ can be assumed as relative magnitude of average horizontal strain acted on the structure in hogging. If average ε_h value on the structure is known, $\varepsilon_h/0.0006$ ratio can be calculated. However, in many practical cases, it is very difficult to measure the horizontal strains acted on the structures directly. Also they cannot be separated from the bending related lateral strains. In the cases in which ground movements are assumed to be transferred to the structure without any significant modification, building horizontal strains can be estimated by deriving the free-field ground settlement profile over the pertinent portion of the building. Horizontal strains can then be assumed as 0.5 to 1 times the change in the slope of the free-field ground settlement profile (Boscardin and Cording, 1989). Herein, it has to be noted that the results will be conservative.

For the known values of Δ/L , C_b and C_d and the estimated/measured value of $\varepsilon_h/0.0006$, a new value of ε_t is calculated through equations 3.3 and 3.4. The corresponding damage class is then determined according to the new value of ε_t . In bending critical cases where the estimated/measured ε_h exceeds the $\varepsilon_{crit} = 0.06\%$ ($\varepsilon_h/\varepsilon_t \geq 1$) and in diagonal tensile strain critical cases where $\varepsilon_h > 0.085\%$, the damage class can be approximately determined by comparing ε_h directly to limiting tensile strain boundaries in Table 2.3. In other words, horizontal strain in those cases is assumed to be directly responsible for the prospective building damage. In Figure 3.2, the variation of C_b^* and C_d^* coefficients for varying L/H values is illustrated. Note that these curves are derived for a masonry wall with $E/G = 2.6$ situated in hogging. As shown, as $\varepsilon_h/\varepsilon_t$ increases (role of ε_h on prospective damage increases), the values of C_b^* and C_d^* are reduced. The reduction of C_b^* is higher than that of C_d^* .

because ε_h is working in parallel to the longitudinal strains due to bending. In addition, it should be noted that, as $\varepsilon_h/\varepsilon_t$ increases, bending critical range of L/H slightly increases because tensile strains in the longitudinal direction (due to bending and imposed horizontal strain) are increased. Note that a reduction of C_b^* and C_d^* results in an increase of the calculated new ε_t value and accordingly damage class for a given Δ/L .

It is emphasized that the mentioned modifications to incorporate the effect of horizontal strains are performed basically for making evaluations for deep beam equations, as will be presented in the next subsection. The proposed modifications are not intended to constitute a new in-situ assessment methodology.

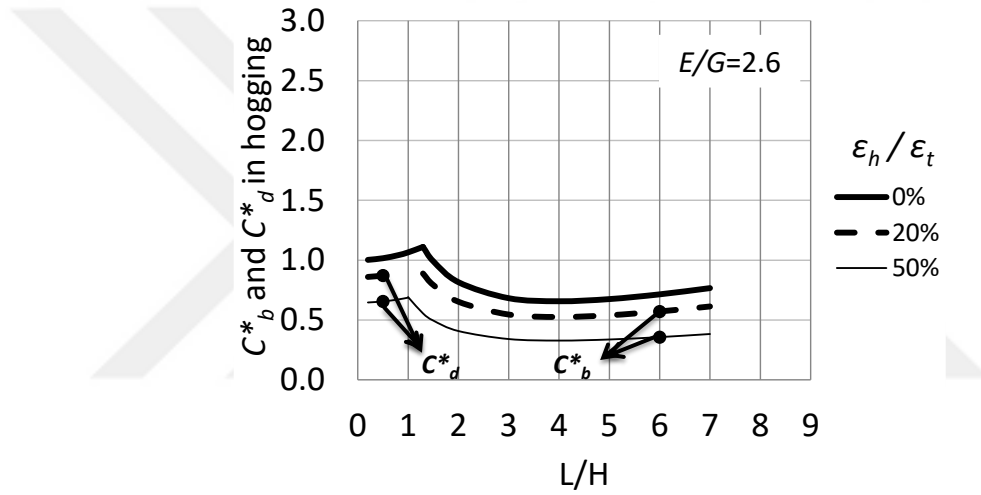


Figure 3.2 : Variation of C_b^* and C_d^* functions with L/H .

3.1.3 Back-calculation of the coefficients of the LTSM transfer functions from real and simulated case studies

Using the back-calculated coefficients of transfer functions and utilizing the implementation of the effect of horizontal strain, a simple evaluation of available data is conducted here. For this purpose, the real case studies of Boscardin and Cording (1989) (case 1) and Farrell et al. (2014) (cases 7 and 8), the experimental data of Son (2003) (case 2) and Giardina et al. (2012) (cases 3-6) and numerical test data of Giardina et al. (2014) (cases 9-11) are used (Table 3.2). The specific intervals for C coefficients are then back-calculated to compare with C coefficients derived by the extracted deep beam equations in Table 3.1. Case 1 is a four-storey brick bearing wall structure subjected to ground movements due to the excavation of a twin tunnel. The foundation consists of rubble strip footings. An extensive shear damage was observed

in the hogging part of one of the walls over a length which is almost equal to the wall height ($L/H \approx 1$). Cases 7 and 8 refer to a two-storey brick bearing wall structure overlying a tunnel excavation. It also has strip footings. It was reported that this building responded relatively flexible to tunnelling-induced ground movements. Moderate level of damage was observed in the sagging and hogging parts of the building. Due to the complex nature of real site problems and lack of data reported, very few real case studies were available to make back-calculations. On the other hand, as being in a controlled environment, the measurements and results of laboratory tests and numerical tests offer advantages for the theoretical robustness of the back-calculation work. The cases 2-6 are therefore derived from experimental studies (Son, 2003 and Giardina et al., 2012) including physical tests of a 1/10-scaled brick masonry façade with openings under artificial hogging conditions. Shear-related damages were observed for these cases. Note that the cases 3-6 are sub-steps of the wall experiment of Giardina et al. (2012). The last three cases (9-11) are the simulations created using the proposed damage functions of Giardina et al. (2014) in which many parametric nonlinear finite element analyses were conducted. All simulated cases include a brick masonry wall model which is similar in size to the wall physically tested in Giardina et al. (2012). The parametric damage functions allow the application of different opening ratios, different magnitude of Δ/L and sagging conditions.

Table 3.2 : The reported features of the case studies.

Case reference and number	Reported data						Damage state and max. crack width (mm) at $(\Delta/L)_{meas}$	Observed damage class
	L/H	E/G	$(\Delta/L)_{meas}$	ε_h	$\frac{\varepsilon_h}{0.0006}$			$\varepsilon_{lim(low)}$ $\varepsilon_{lim(up)}$
Boscardin and Cording (1989)	1	1.00	2.60	Hogging 5.6×10^{-3}	7.5×10^{-3}	12.5	Severe diagonal cracking near to window openings	Severe (0.003, higher)
Son (2003)	2	2.00	12.9 ^a	Hogging 0.53×10^{-3}	1.7×10^{-3}	2.83	Diagonal cracks around 10 mm	Moderate (0.0015, 0.0030)
	3	1.21	11.0	Hogging 6.00×10^{-4}	0	0	Hairline cracks	Negligible (0.0000, 0.0005)
	4	1.21	11.0	Hogging 1.30×10^{-3}	0	0	Many cracks from 5 to 15 mm	Moderate (0.0015, 0.0030)
Giardina et al. (2012)	5	1.21	11.0	Hogging 1.90×10^{-3}	0	0	Many cracks from 15 to 25 mm	Severe (0.003, higher)
	6	1.21	11.0	Hogging 2.60×10^{-3}	0	0	Many cracks larger than 25 mm	Very severe (0.003, higher)
	7	2.75	2.60	Sagging 1.90×10^{-3}	-0.32×10^{-3}	N/A	Cracking	Moderate (0.0015, 0.0030)
Farrell et al. (2013)	8	0.42	2.60	Hogging 0.6×10^{-3}	0.53×10^{-3}	0.88	Severe cracking	Moderate (0.0015, 0.0030)
	9	1.21	2.60	Hogging 2.60×10^{-3}	0	0	-	Slight (0.075, 0.150)
Giardina et al. (2014)	10	1.21	8.00	Hogging 2.60×10^{-3}	0	0	-	Moderate (0.0015, 0.0030)
	11	1.21	11.0	Sagging 2.60×10^{-3}	0	N/A	-	Very severe (0.003, higher)

Note: ^a This value was extrapolated by using the opening ratio - E/G relationships given by Son and Cording (2007) for a shear/normal stiffness ratio of 1/8 for the joints.

Table 3.2 also lists the structural properties and deformation measures for each case, as well as the observed damage classes. The E/G column shows the values which are explicitly reported (for cases 3-6), extrapolated (for cases 2 and 9-11) and assumed (for cases 1, 7 and 8). A similar extrapolation technique (based on the suggestions of Son & Cording, 2007 and Giardina et al. 2015b) that is used for the cases 2 and 9-11 cannot be done for the cases 1, 7 and 8, due to missing information regarding the geometrical properties and opening ratios of the walls. However a small sensitivity study is conducted using different E/G values in the back-calculation process of these

cases. In Table 3.2, $(\Delta/L)_{meas}$ represents either the ultimate deflection ratio for the cases 1, 2, 7, 8, 9, 10 and 11 or the deflection ratios attained at the end of various sub-steps in the experiment of Giardina et al., 2012 (cases 3-6). Note that Δ/L and the corresponding damage increased progressively during this experiment.

The effect of horizontal strains is also considered for the hogging cases 1, 2 and 8. While the horizontal strains in the cases 1, 7 (sagging) and 8 are calculated with the practical assumption that they vary between 0.5 and 1 times the change in the slope of the free-field ground settlement profile, in case 2 it was directly measured on the specimen wall. In the cases 3-6, no horizontal strain was considered by the authors due to the use of a lubricated rubber interface between the test wall and supporting steel profile. Likewise, in the cases 9-11, horizontal strains are assumed to be zero because of the presence of a smooth interface at the wall base in the numerical analyses.

Damage classes were either explicitly reported based on the visual inspection results (cases 1, 7 and 8) or defined based on the maximum crack width and crack intensity (cases 2-6). For the cases 9-11, damage classes are directly defined as a result of the damage functions. Note that the maximum crack widths listed for the cases 2-6 are ten times the experimentally observed values to be in line with the field scale (Son, 2003 and Giardina et al., 2012).

Based on the reported data and damage classes, the evaluation procedure is performed as follows:

- C_b and C_d coefficients of the transfer function are calculated based on reported L/H and E/G values through the extracted original deep beam equations given in Table 3.1. The minimum of C_b or C_d is decisive on the dominant damage type (bending cracking or diagonal tensile cracking). The defined damage type is compared to observed damage type. For all of the cases presented here, the defined damage type is in line with the observed damage type.
- The range of $C_{(low)}$ and $C_{(up)}$ is back-calculated using equations 3.5 and 3.6 substituting $(\Delta/L)_{meas}$ and $\varepsilon_{lim(low)}$ and $\varepsilon_{lim(up)}$ boundaries which correspond to the reported damage class. Note that back-calculation is based on only measured and observed case information and thus independent of the deep beam equations.

$$C_{(low)} = \frac{(\Delta/L)_{meas}}{\varepsilon_{lim(up)}} \quad (3.12)$$

$$C_{(up)} = \frac{(\Delta/L)_{meas}}{\varepsilon_{lim(low)}} \quad (3.13)$$

- The obtained back-calculated range $C_{(low)}$ - $C_{(up)}$ is then compared to calculated C_b or C_d value, whichever is critical (minimum). In the hogging cases, if exists, the effect of horizontal strain can also be considered. Note that incorporation of the horizontal strain will result in a reduction in the coefficients of the transfer functions (see equations 3.5 and 3.6).

3.1.4 Discussion of the results of performed back-analysis

This section presents and discusses the results obtained through the back-calculation of the C coefficients from the 11 case studies. Figure 3.3 compares the calculated C_b and C_d coefficients to back-calculated $C_{(low)}$ - $C_{(up)}$ ranges for each case presented in Table 3.2. It should be stated that, for an ideal agreement between observed damage & measured crack widths and attributed limiting tensile strain boundaries, the calculated C_b and C_d coefficients are expected to locate in-between the back-calculated $C_{(low)}$ and $C_{(up)}$ boundary values. From this aspect, a general agreement between calculated C_b and C_d coefficients and back-calculated ranges is seen for the majority of the examined cases.

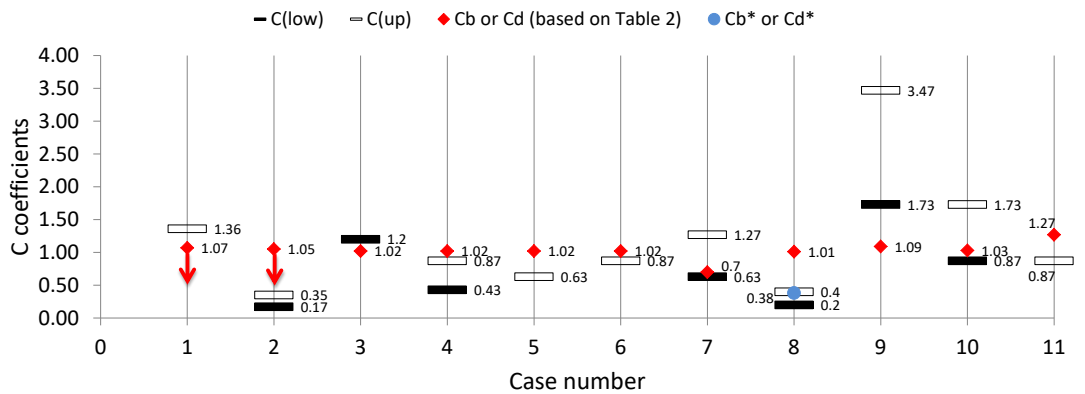


Figure 3.3 : Comparison between calculated decisive C values and back-calculated $C_{(low)}$ - $C_{(up)}$ ranges.

For the cases 2, 5, 8 and 11 in which moderate or higher damage was observed, the calculated decisive C_d coefficients tend to clearly overestimate the back-calculated C

ranges. This might be because calculated C_d is a constant value and it cannot capture degraded stiffness and damage localizations beyond moderate damage of these structures. On the contrary, in the cases 3 and 9 in which negligible or slight damage was observed, the calculated decisive C_d coefficients tend to clearly underestimate the back-calculated C ranges. This might be because these structures response actually in a stiffer manner than being estimated by the deep beam approach.

Recalling the generic form of the transfer functions, $\Delta/L = C\varepsilon_{crit}$, it can be stated that overestimation of C_b or C_d will yield an underestimation of the damage for the structures having moderate or higher damage in reality. This situation can also be considered in the assessment procedure of the structures which are damaged prior to excavation works. However, being quite conservative as a result from the direct application of free-field ground settlements, the mentioned underestimation is most probably compensated during the evaluations made by LTSM.

For the cases 1, 4 and 6, in which moderate to very severe damage was observed, the calculated decisive C_d values tend to approach to the upper boundary of the back-calculated C ranges. In the cases 7 and 10, in which moderate damage was observed, the decisive C_b (for case 7) and decisive C_d (for case 10) tend to approach the lower boundary of the back-calculated C ranges. Note that case 7 corresponds to a bending critical case and thus C_b is decisive.

It has to be highlighted that the varying E/G values from 2.6 to 10 for the cases 1 and 8 has insignificant effect on the results of the evaluations made above. This is because the influence of the value of E/G on the determination of the decisive C_d coefficient is quite limited, especially for the hogging cases up to $L/H=2$ (see Figure 3.1 (b)). Note that the decisive C coefficient shifts from C_b to C_d for the values of $E/G \geq 11$ for case 7. This situation could have an effect on the evaluation of case 8.

There are only three cases (1, 2 and 8) for which the effect of horizontal strain can be incorporated. For the cases 1 and 2 the magnitude of the average horizontal strain ε_h exceeds 0.085%. Therefore, equation 3.4 cannot be used. However, it is obvious that calculated C_d value would be lower for these cases. The arrows facing downward represent this expected reduction in the calculated C_d value (Figure 3.3). Interestingly, the damage class of these two cases (severe for case 1 and moderate for case 2) can be determined by means of a direct comparison of the average horizontal strain to the

limiting tensile strain boundaries $\varepsilon_{lim(low)}$ and $\varepsilon_{lim(up)}$. This situation might show that damage in these cases is governed by mainly horizontal strains. For case 8, in which average horizontal strain $\varepsilon_h < 0.085\%$, the overestimating C_d value (1.01) can be reduced to a C_d^* value (0.38) by using equation 3.4. As seen in Figure 3.3, the reduced C_d^* value provides a better approximation to the back-calculated range.

3.1.4.1 Crack localizations leading to underestimated predictions made by the LTSM

The results of the experimental study of Giardina et al. (2012) (case 6 in the previous sub-section) showed that crack localizations occurred around the large wall openings in the later stages of the building settlement can be reason of the underestimated predictions in the LTSM. The details of this setup can be found in Appendix A. The authors applied the LTSM to predict the final damage class and compared it with the damage class determined experimentally at the end of the test. Although a relatively large value of $E/G = 11$ is considered to reflect the increased flexibility of the test wall with large openings in the LTSM equations, the damage localizations around the large openings cannot be taken into consideration leading to underestimation of the actual damage class. They reported that while the damage class determined based on the measurements during the wall test becomes very severe, the LTSM predicts moderate level of damage. This result is an example of showing the importance of consideration of the structural details while making settlement damage predictions.

Table 3.3 and Figure 3.4 show the variation of crack width and damage progression during the test. Cracks initialize at the vicinity of the window and door openings and widen throughout the test. It is seen that exact location of these cracks is strongly dependent on the position of the window and door openings. Almost all of the wall damage is limited to these cracks and new crack occurrence at the subsequent stages of the experiment is surpassed due to widening of initial cracks.

Table 3.3 : Crack width at increasing values of applied maximum settlement from straingage measurements (modified from Giardina et al., 2012).

Maximum applied settlement (mm)	Crack 1 (mm)	Crack 2 (mm)	Crack 3 (mm)	Crack 4 (mm)	Crack 5 (mm)	Crack 6 (mm)
3.0						
3.5		0.261	0.630			
4.0		0.492	0.367	0.044		
4.5		0.757	0.561	0.097		
5.0		0.986	0.714	0.134	2.080	
5.0		1.118	0.804	0.150	2.133	
5.5		1.339	0.945	0.197	2.255	
6.0		1.556	1.107	0.227	2.360	
6.5		1.800	1.261	0.269	2.471	
7.0		2.019	1.419	0.312	2.570	
7.5		2.261	1.565	0.348	2.566	
7.5		2.577	1.790	0.404	2.788	2.154
8.0		2.852	1.985	0.448	2.904	2.235
8.5		3.215	2.217	0.499	3.030	2.418
9.0		4.269	2.875	0.641	2.370	3.664
9.5	5.000	3.909	3.665	0.851	1.796	3.134
10.0	6.300	3.826	3.803	0.891	1.766	3.069
10.5		3.842	3.935	0.939	1.743	3.042
11.0	9.500	3.888	4.048	0.975	1.750	3.041
11.0	10.480	3.897	4.128	0.998	1.733	3.024
11.5		3.950	4.219	1.032	1.731	3.014

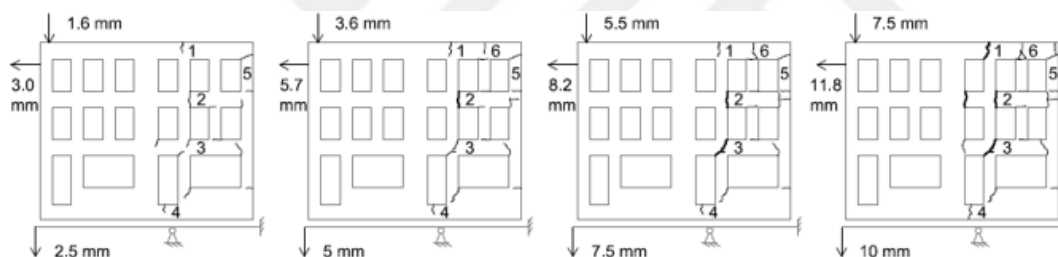


Figure 3.4 : Crack progress and global displacements in response to applied increasing settlements.

When the test was stopped at the applied settlement of 10 mm, attained maximum crack width (crack 1 in Figure 3.4) was 10.5 mm pointing out a very severe damage class when the scale of the specimen (1/10) is considered. In Appendix A, calculations made for the analytical assessment of this specimen through the LTSM are presented.

4. A TWO-STAGE NUMERICAL ANALYSIS APPROACH FOR THE ASSESSMENT OF THE SETTLEMENT RESPONSE OF THE PREDAMAGED HISTORIC HOCA PASHA MOSQUE

4.1 Introduction

Well-documented case studies in which different types of excavation, soil and building conditions are examined offer a great resource to understand complicated interaction problem between excavation-soil and building. A number of case studies representing various circumstances from different countries have been published (Burland et al., 2011; Finno et al., 2002; Bryson and Kotheimer, 2011; Korff et al., 2012; Amorosi et al., 2014; Fu et al., 2014; Pujades et al., 2015). In many studies (Burd et al., 2000; Pickhaver et al., 2010; Losacco et al., 2014; Giardina et al., 2015a; Fagnoli et al., 2015), finite element simulations are used with different levels of sophistication of the geometrical and material modelling. In some numerical studies (Giardina et al., 2015a; Son and Cording, 2011), the effect of initial building damage is also considered. However, due to lack of detailed information and other uncertainties regarding excavations, foundations, soil properties and structures, a series of assumptions and simplifications are required to simulate the actual case, which are difficult to be verified.

This chapter presents another case study in which the current LTSM underestimates the final damage class. The focus of the study is on the examination of the settlement response of historic Hoca Pasha Mosque with large window and door openings in its masonry load-bearing walls. In addition, several uncertainties within the case are shown along with the considered assumptions and simplifications. Table 4.1 characterizes the present case with regard to excavation and construction works, soil, building deformation, building structure and pre-existing damages. It indicates the complexity and the availability of (monitoring) data for each aspect. Typical for this case study is that the initial damage is relatively well known; the building has clearly localized cracks, which have been monitored during the excavation works. By contrast, the excavation history is relatively complicated.

The originality is to simulate and assess the settlement response of the pre-damaged historic Hoca Pasha Mosque using a two-stage numerical analysis approach. According to this, firstly the structural model with and without pre-existing cracks is verified in a way that each façade wall is loaded separately using the measured settlements and taking into account flange affects (stiffness contribution) of other connecting walls. This stage to a large extent reduces modelling uncertainties. In the second stage, the effects of other parameters, i.e. the building stiffness, bedding stiffness and combined settlement field due to sequential excavation works are investigated. This naturally increases the modelling uncertainties, but it more closely resembles the practice of *predicting* settlement responses. Although the mentioned stages are not dependent directly on each other, the staging approach helps to verify the building model. Besides an examination of the specific case, more generic results are obtained through parametric analyses performed in the second stage.

Table 4.1 : Characterizing the Hoca Pasha Mosque case study in terms of its geotechnical and structural complexity and the availability of (monitoring) data.

Aspect	Complexity level	Data availability level
Excavations activities	High	Detailed (through the continuous monitoring data and reports regarding tunnel and shaft construction and supporting)
Soil structure	Moderate	Standard (through the reports derived from the basic in-situ soil tests)
Building deformation (building settlement and crack progress)	Moderate	Detailed (through the continuous settlement and crack monitoring data)
Building structure (geometry and material properties)	Low	Standard (through the in-situ drawings, previous expertise and publications on similar historic structures in the region)
Existing damages (crack locations and initial crack severity)	Moderate	Detailed (through the conditional building survey)

4.2 Description of the Mosque and the Neighbouring Excavation Works

A comprehensive railway transportation project, namely Marmaray, was implemented in Turkey between 2006 and 2013. As part of this project, intensive tunnel and shaft excavations were carried out in the most historic parts of Istanbul.

In the Hoca Pasha district, 135 buildings were inspected in total. Fifty eight out of 135 buildings were considered to be of moderate or high risk. These buildings were re-investigated more precisely in the detailed assessment stage. Furthermore, during the subsequent stages of the tunnel and shaft construction, evaluations were revised for

some of these buildings according to increasing displacements, developing damages and the state of excavation. The Hoca Pasha Mosque as a listed historic structure was one of these buildings affected by the shaft and tunnel excavations of the Sirkeci Station in the Hoca Pasha district.

The Hoca Pasha Mosque was rebuilt in 1868 replacing the previous one destroyed by a fire (Sefer and Ahunbay, 2015). As seen in as-is drawings shown in Figure 4.1 (a), the main structural system of the mosque relies on plastered 900 mm thick load bearing masonry walls made of solid bricks and mortar joints. An outbuilding with a separate reinforced concrete frame system was built next to the main building (Figure 4.1 (a)). The mosque has large arched windows ($w \times h$: 1840×3860 mm) in all of the façades and doors in the northwest (NW) and northeast (NE) façades (Figures 4.1 (b), (c) and (d)). An internal wall of almost half of the storey height divides the narthex and prayer hall. The mosque is actually a one-storey building, but there is a mezzanine floor inside. The mezzanine floor, that is constructed with steel and timber joists, and the roof are both supported by slender steel columns, situated inside the mosque, and the external masonry walls. The overall wall height is 8.25 m from the wall bottom to the eave. During the examinations, any particular foundation system was not observed except for the main walls extending approximately 1 m downwards into the ground as common as in the masonry buildings constructed in the same era. On the other hand, it is assumed that the single footings are present under the internal steel columns.

The location of the mosque was considered as critical due to closeness of intensive excavation works carried out during the construction of the Sirkeci underground station. The location of the mosque and nearby excavations with excavation advancement directions are illustrated in Figure 4.2 (a). Monitoring results revealed that the mosque was affected to a varying extent by the indicated elliptic ventilation shaft (WVS), the pilot tunnel (NPLT) in the north platform tunnel (NPF) and the north and south ventilation chambers (NPFV and SPFV, respectively). It was observed that other excavations, namely the south platform tunnel excavation (SPF), the enlargements in north & south platform tunnels (NPF and SPF, respectively), the centre walkway tunnel excavation (CE), the short connection excavations (CNVs) between NPF and SPF, and the upwardly inclined south entrance tunnel excavation (ISL) had no or less impact on the settlement response (Monthly Monitoring Reports, 2008-2013) (Figures 4.2 (a) and 4.3).

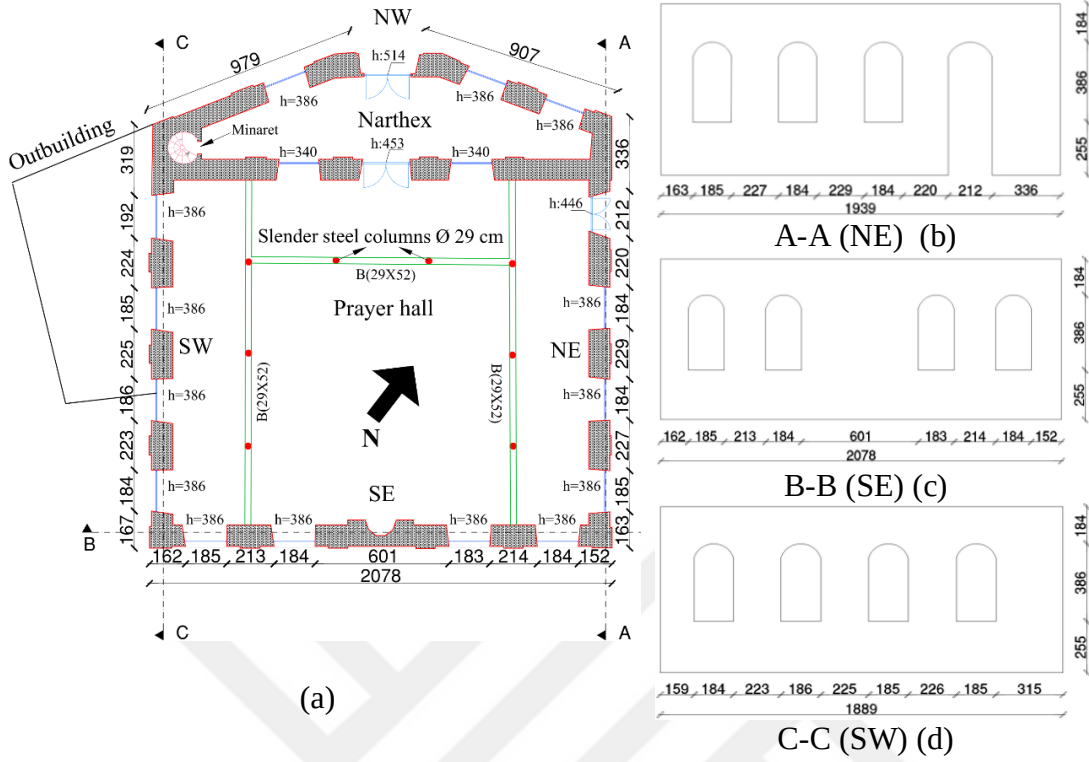


Figure 4.1 : (a) As-is drawing of the ground floor elevation of the Hoca Pasha mosque and (b,c,d) section views (dimensions in cm) (Akgun, D., 2016).

The WVS excavation wall approximates the southeast (SE) façade at 10m. The shaft diameter is 26.4 m on the long axis and 22.8 m on the short axis. The entire shaft depth is 58 m. The first 21 m of the shaft excavation (the upper shaft) was constructed using a secant pile wall technique. For this purpose, grouted and reinforced concrete piles with 900 mm diameter were consecutively cast on the perimeter of the elliptic shaft. Then, their tips were capped with a rigid reinforced concrete ring beam (b×h: 1000×1500 mm). As the excavation proceeds, piles were connected circumferentially by reinforced concrete ring beams (b×h: 1000×1000 mm) cast at every 1500 mm. The ring beams significantly increase the radial rigidity of the circular shaft. The remaining 37 m of the excavation (the lower shaft) was carried out in the bedrock as a stepwise open face excavation, supported by a thick wall consisting of wire meshed shotcrete and HEB 200/100 steel profiles, fixed via rock bolts. No internal cross strutting was used at any stage of the shaft excavation, not to block the equipment access through the shaft (Method Statements for Excavation and Support Works, 2008).

The excavations of the tunnels (NPF, SPF, hereinafter referred to together as PFs, and CE) and the ventilation chambers (NPFV and SPFV, hereinafter referred to together

as PFVs) were carried out using the New Austrian Tunnelling Method (NATM), approximately 50 m beneath the ground surface. The PFVs have the largest cross-section with 17 m diameter among all tunnels excavated for the Sirkeci Station. The length of the PFVs is 35 m. While the NPFV partly under-passes the mosque at a skewed angle, the centre line of the SPFV is at a 40 m in plan distance from the closest corner of the mosque (Figures 4.2 (a) and 4.3). In order to increase the face stability and limit deformations, tunnel cross-sections larger than 50 m² (as the PFs and PFVs) were divided into sub-sections (Figure 4.2 (b)). Then, each sub-section was excavated sequentially starting from the pilot tunnel sub-section (respectively succeeded by the excavation of the top heading, the medium heading, the bench cut and the invert). The NPLT, which became quite influential on the mosque settlements, was actually a sub-section excavation of the NPF having a 27 m² cross-sectional area (Method Statements for Excavation and Support Works, 2008).

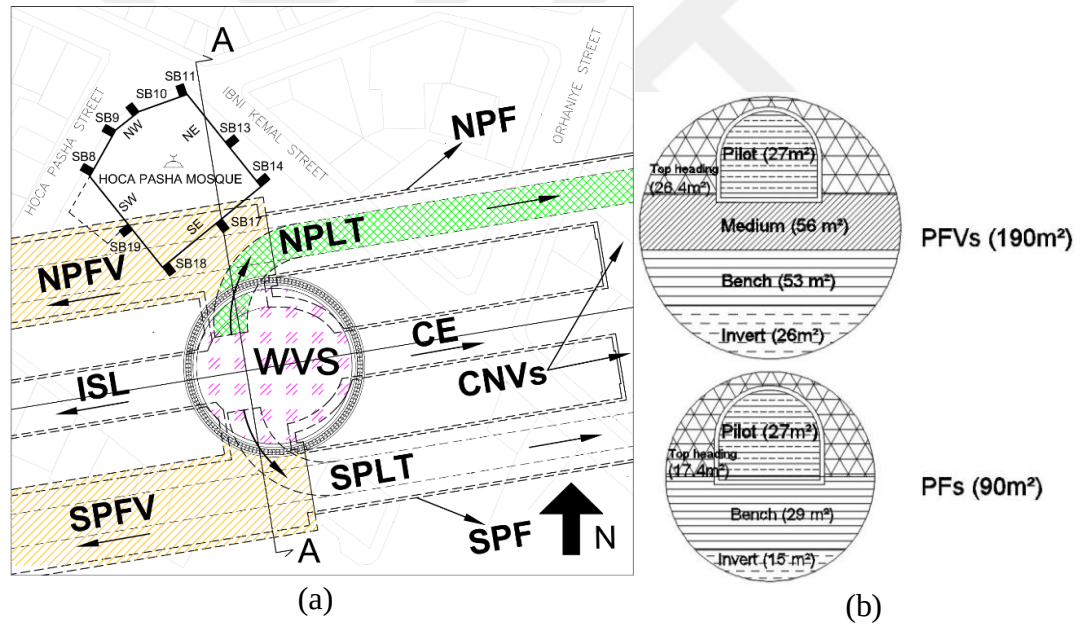


Figure 4.2 : (a) Location and neighbouring excavation works (b) Tunnel cross-sections (modified from Method Statements for Excavation and Support Works, 2008).

Table 4.2 presents starting and completion dates of the neighbouring excavations. Observations regarding the influence of these excavations on the mosque behaviour are listed in the last column. Before the tunnel and ventilation chamber excavations started, the WVS excavation had been completed on 31 July 2008. After taking down the excavation machines through the WVS, the pilot tunnelling works in the PFs (SPLT and NPLT, respectively) started from the inside of the WVS. After following a

curvilinear route, they proceeded eastwards, moving away from the WVS and accordingly from the mosque (Figure 4.2 (a)). The sequentially enlargement excavations were then started for SPLT and NPLT excluding the curvilinear sections near to the mosque. Afterwards, the pilot tunnel excavation backward into the SPFV commenced and it was followed by the enlargement excavations in this section. Finally, the pilot tunnel and enlargement excavations in the NPFV were completely carried out. The difference in timing of the excavation of the WVS, NPLT and NPFV excavations particularly offers an opportunity to investigate the accumulating effects on the mosque.

Table 4.2 : Starting and completion dates of neighbouring excavations.

Excavation name (by sequence)		Starting date	Completion date	Observed influence on the mosque
WVS	Secant piled wall installation + Upper shaft excv. (21 m) + archaeological surveys	February 2006	February 2008	Negligible impact. Settlements up to max. 1.5 mm at the southeast façade
	Lower shaft excavation (37 m)	February 2008	July 2008	Influential from start to the end
SPF (SPLT and other sub-section enlargements)		November 2008	June 2010	No influence, neither from pilot nor from enlargement excavations
NPF (NPLT and other sub-section enlargements)		December 2008	August 2010	Substantial influence, especially from the curvilinear part of the pilot excavation
SPFV	Pilot	January 2010	February 2010	The pilot tunnel excavation was not influential
	Sub-section enlargements	March 2010	July 2010	The enlargements were influential
NPFV	NPFV pilot	April 2010	July 2010	Substantial influence for all excavation stages
	Top heading	August 2010	October 2010	
	Medium	October 2010	December 2010	
	Bench & Invert	December 2010	January 2011	

4.3 Local Site Condition

Borehole results revealed that the underlying geological structure in the vicinity of the mosque consists of three main layers (Geological Report, 2008; Pressuremeter Test Results, 2009). For the A-A section, indicated in Figure 4.2 (a), the obtained soil profile is shown in Figure 4.3. The first 15 m from the ground surface is man-made fill, consisting of sand, gravel and clay (I). Measurements made in surrounding wells (in 2013) showed that the ground water table is also around 15 m beneath the surface. The second layer (II), which is not thicker than 10 m, is composed of clayey silty sand with shell fragments. The third layer (III) is identified as a greywacke mass, comprised of mudstone, sandstone and claystone, which are the components of the Trakya Formation. The sections shown with IV are identified as diabase.

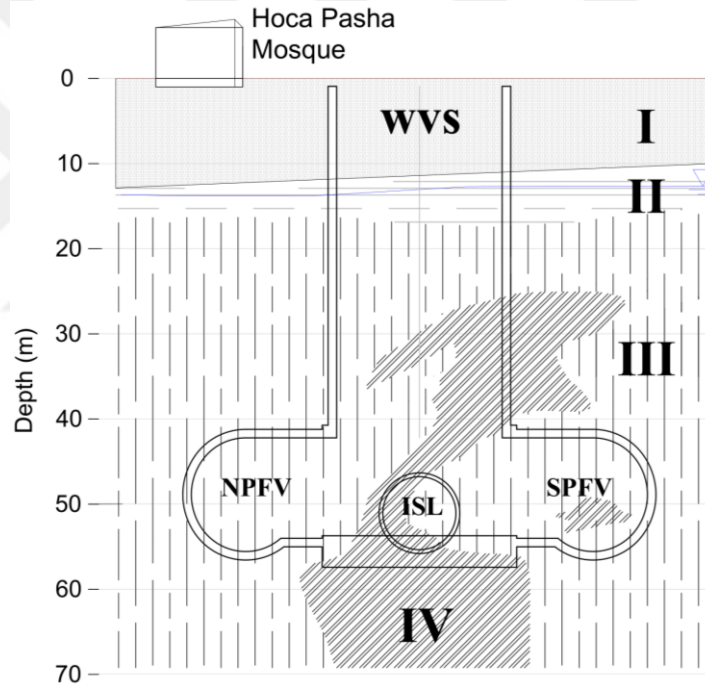


Figure 4.3 : Sectional view of the excavation site and geological sections (A-A section in Figure 4.2 (a)) (modified from Method Statements for Excavation and Support Works, 2008).

The mechanical properties of the main geological layers are listed in Table 4.3. In this table, E_s , ν , γ_s , c and φ show the Young's modulus, Poisson's ratio, volumetric weight, cohesion and angle of internal friction, respectively. Note that these are the values reported in the borehole test reports (Pressuremeter Test Results, 2009).

Table 4.3 : Reported mechanical properties of the geological layers (Geological Report, 2008; Pressuremeter Test Results, 2009).

Soil Layer	Depth (m)	E_s (MPa)	ν	γ_s (kN/m ³)	c (kN/m ²)	ϕ (degree)
Man-made fill (I)	0-15	33.0	0.30	19.0	0.00	30.0
Clayey silty sand (II)	10-15	56.0	0.30	19.0	0.00	34.0
Greywacke (III)	15 to deeper	150	0.35	21.0	200	30.0

4.4 Monitoring Results and Observed Response

An intensive monitoring program was performed, starting from 2008 to the end of 2013. Settlements of the mosque were measured via levelling instruments by reading the vertical displacements of the monitoring bolts (SB monitoring points in Figure 4.2 (a)) attached to the external walls. Because of the absence of an effective foundation system to inhibit or reduce the ground movements which will be transferred to the walls and by means of the placement of the monitoring bolts to the bottom of the walls, just few centimeters away from the ground surface, any significant modification in the settlement readings would not be expected. Free-field ground settlements could not be measured effectively due to building congestion and traffic in the region. Instead, estimated free-field settlements are used in the analyses to be explained in the following sections.

The settlement time history taken from SB monitoring points are shown in Figure 4.4. Excavation and construction activities, which are influential on the settlement response of the mosque, are also marked in this figure. Note that the settlements that occurred during the secant piled wall installation and the upper WVS excavation (i.e. the cut and cover excavation of the first 21 m) are not included in this figure. However, the monitoring reports (Monthly Monitoring Reports, 2008-2013) show that the settlements at that stage were negligible (≤ 1.5 mm).

As expected, settlement readouts measured on the southeast (SE) (where SB14, 17 and 18 were attached) and southwest (SW) (where SB8 and 19 were attached) façades which were relatively closer to the WVS, NPLT and NPFV excavations have been higher than the other readouts. After the completion of the WVS excavation, the rate of measured settlements of the mosque decreased, until the pilot tunnel excavations

SPLT and NPLT started. Sudden increases in the slopes of the settlement versus time curves point out that the NPLT excavation, specifically the curvilinear section, has become quite influential. Between February 2009 and April 2010, an increase presumably due to CE pilot tunnel excavation and enlargement excavations in the PFs was observed. From April 2010 onwards, the settlements significantly increased again. This time, the reason was the pilot tunnel and enlargement excavations in the PFVs. It should be particularly highlighted that the pilot tunnel and the enlargement excavations in the NPFV played a major role in this increase. In order to limit the settlements at that stage, two ground stabilizations and a following uplifting application were carried out. The first ground stabilization attempt was a cement injection procedure performed horizontally from the inside of the WVS towards the mosque. In addition to cement injection, horizontal steel rods were driven into the soil in order to compact the ground under the mosque. The second attempt was aiming at both ground stabilization and uplifting the severely settled parts of the mosque. For this purpose, vertical steel rods, 7 m long and 36 mm in diameter, were driven into the soil with an inclined angle at every 40 cm around the main external masonry walls. After performing a primer polyurethane (PU) injection, uplifting was performed through a secondary and deeper PU injection. Consequently, an upward movement (up to 3 mm) around SB13, 14 and 19 was obtained. The effect of uplifting can be seen in the settlement time history of SB13 in Figure 4.4 between September and November 2010. All of the excavation works in SPFV and NPFV tunnels were finalized towards January 2011.

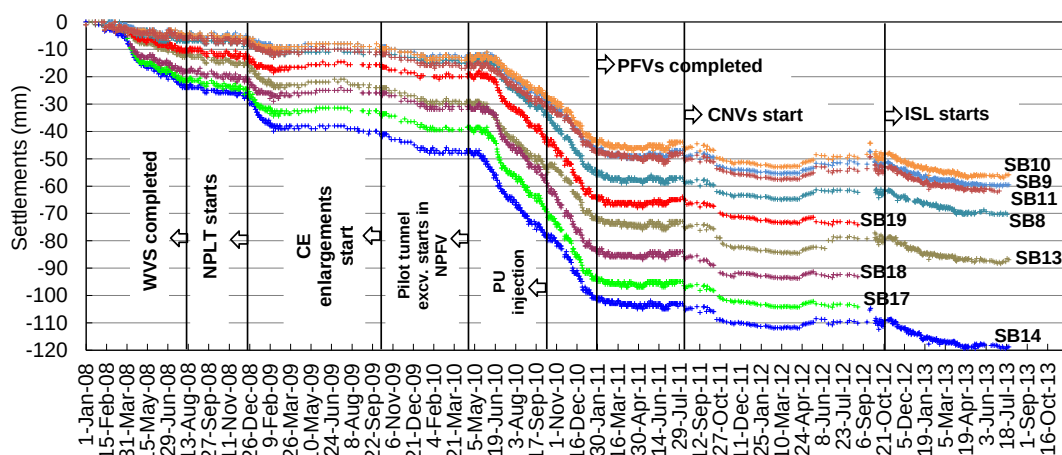


Figure 4.4 : The settlement time-history of the mosque (modified from Monthly Monitoring Reports, 2008-2013).

4.4.1 Conditional survey prior to excavations

A conditional survey conducted before the commencement of the excavations revealed the presence of pre-existing localized cracks (C1, C2, C3-1, C3-2, C5, C6 and C10) in all of the façades (see Figure 4.5) (Detail Survey Report, 2008). These cracks all began at the door and window arches, either at the vertices or closer to the bottom of the arches. They have propagated upwards to the eaves almost vertically. The cracks were also visible from the inside of the mosque. While C1 and C2 above the arched door of the NE façade were the widest, C3-2 and C10 above the arched windows in the NW and SE façades were the narrowest cracks. The observed initial width of these cracks varied from 5 to 10 mm. The locations of the pre-existing localized cracks in the ground plan are also given in Figure 4.5. As is, the building was classified as moderately damaged by the inspection team considering its “prior to all excavations” and estimated post condition.

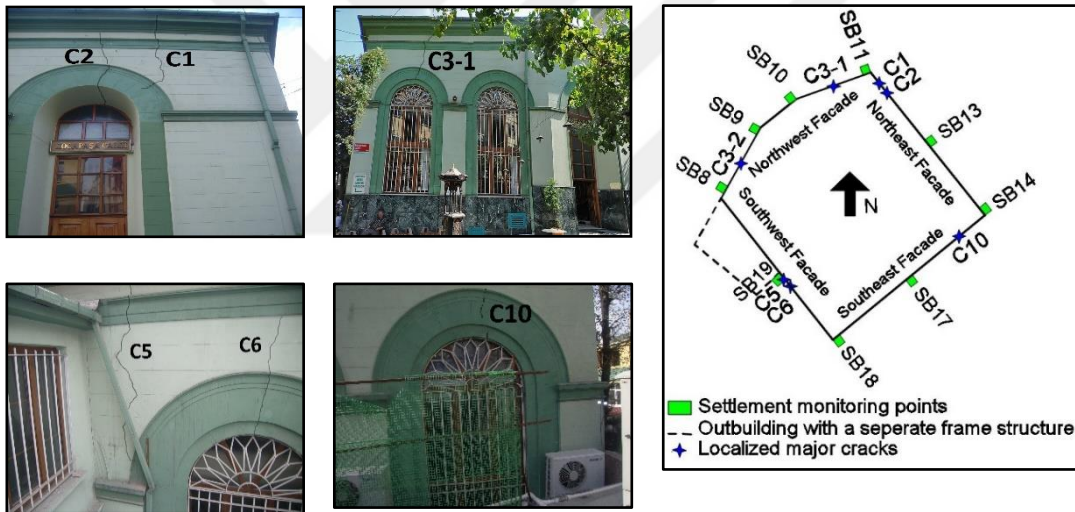


Figure 4.5 : The pre-existing localized cracks and their locations in the plan of the mosque.

4.4.2 Settlement response during the construction and excavation activities

The mosque's response was regularly inspected and reported as the construction and excavation works progress (ITU Evaluation Reports, 2008-2011). At the visual inspection dated 21 January 2009, slight widening of the mentioned pre-existing cracks due to the WVS and curvilinear NPLT excavation were observed. Increases in crack widths were recorded from 12 February 2009 onwards. Two different methods were adopted to measure the crack openings and closures throughout the excavations. Until 23 June 2010, digital callipers were used to measure the crack widths. As of June 2010,

the frequency of crack measurements was increased with regard to increasing settlements due to the NPFV excavations. A separation between the main building and the outbuilding was also observed. Considering the continuing NPFV excavations, from 23 June 2010 onwards, more precise scale type crack meters were bonded and used to measure the crack widths. Although a few new cracks formed around the localized pre-existing cracks during the NPFV excavations, their widths remained relatively insignificant (none of them exceeded 0.9 mm until the end of the excavations).

Figure 4.6 shows the variation of the widths (crack activity) of the major pre-existing localized cracks. Although a trend cannot be given for the initial data obtained by the digital callipers for the period before 23 June 2010, the final digital calliper readouts are added to the data obtained by scale type crack meters. No measurements for crack C3-2 were carried out.

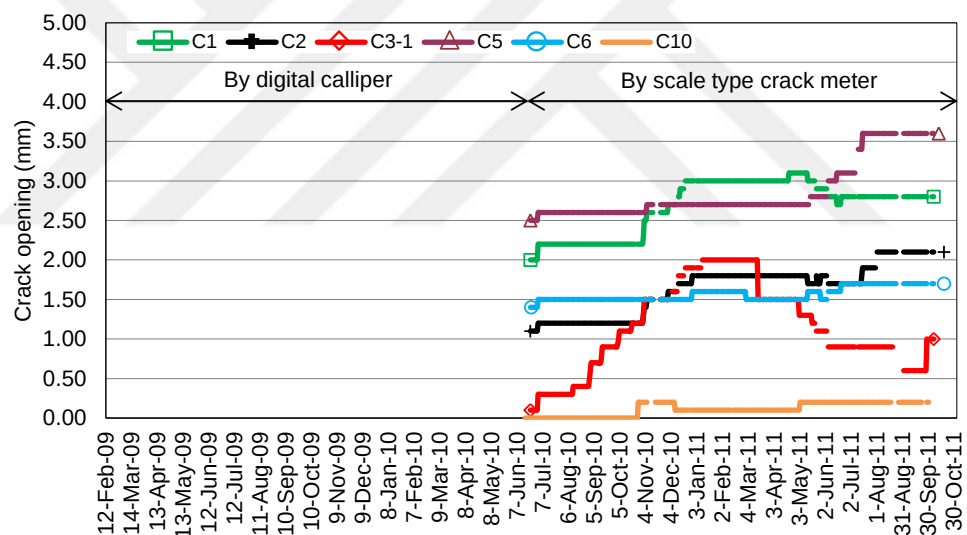


Figure 4.6 : Opening and closures of the pre-existing localized cracks.

Figure 4.6 shows that crack C3-1 widens and closes significantly during the NPFV excavations. As the mosque settlements slow down towards March 2011, a closing is observed for this crack. Another increase in the activities of C1 and C2 is noted during the top heading and medium sub-section excavations of NPFV. The C5 and C6 cracks seem to be influenced least by the NPFV excavations. However, an increase in their widths is observed as of June 2011 possibly due to the CNV excavations. Crack C10 shows minor variation throughout the excavations.

4.4.3 Evaluation of the Mosque's response using empirical limits of conventional deformation measures and the LTSM

4.4.3.1 Empirical evaluation

Monitored settlement response of each external masonry wall is examined using conventional deformation measures, namely angular distortion, deflection ratio and rigid rotation, and the LTSM. For this purpose, the monitored wall settlement profiles are provided in Figure 4.7 considering seven key dates i.e. after the completion of the WVS excavation (15 September 2008), after the completion of the curvilinear NPLT excavation (24 March 2009), before the NPFV excavations (18 March 2010), after the completion of the NPFV pilot tunnel excavation (29 July 2010), after the completion of the entire NPFV excavations (3 January 2011) and before and after the CNV excavations (21 July 2011 and 1 March 2012). As seen, while the SE façade experiences sagging (a convex settlement profile), the others experience hogging (a concave settlement profile) throughout the excavations.

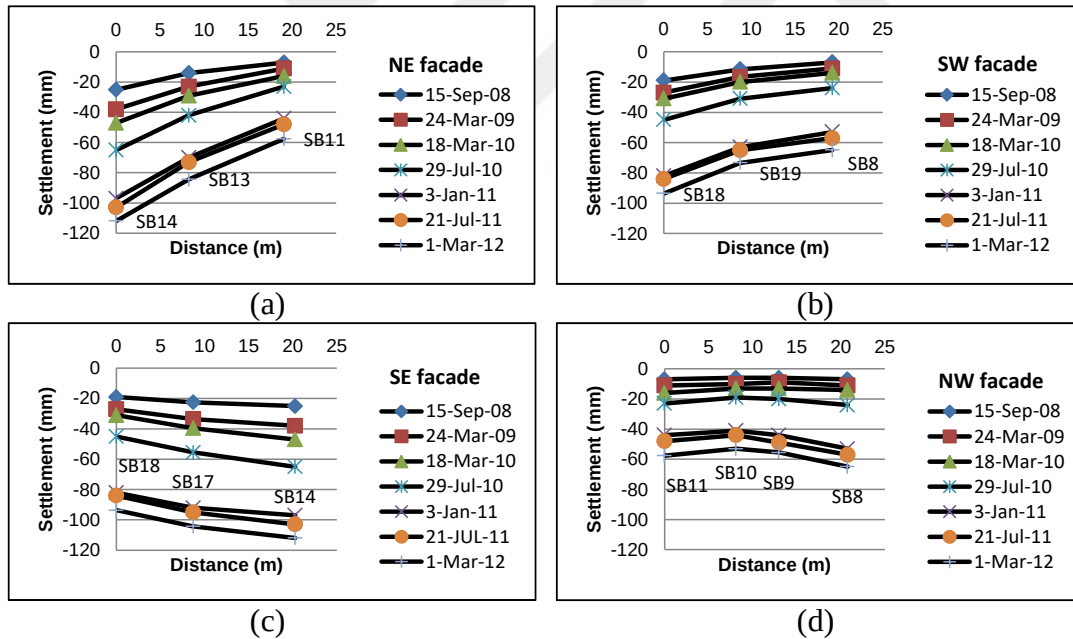


Figure 4.7 : Monitored settlements along the main masonry walls.

Based on the wall settlements at these considered dates, the maximum angular distortion (β_{max}), the deflection ratio (Δ/L) and the rigid rotation (ω) values of the façade walls are calculated (Figures 4.8 (a), (b) and (c), respectively). β_{max} is the largest angular distortion value (β) calculated as the rotation of the straight line connecting two adjacent monitoring points relative to their tilt (Boscardin, 1980). Δ/L is the ratio of relative deflection between two monitoring points to the distance between them

(Burland and Wroth, 1974). ω is the rigid rotation of the façade wall defined by the settlement readouts of two outmost monitoring points. Calculation of these measures are schematically explained in Figure 4.8 (d). Three internationally accepted empirical limit values $\beta_{max} = 0.0005$ (1/2000) (Meyerhof, 1982), $\Delta/L = 0.0003$ (1/3300) (Polshin and Tokar, 1957) and $\omega = 0.002$ (1/500) (Rankin, 1988) are also indicated in Figures 4.8 (a), (b) and (c). The first two values show the cracking limit of unreinforced load-bearing masonry walls in hogging. Note that the $\Delta/L = 0.0003$ value is obtained by interpolating the linear wall length to wall height ratio (L/H) versus Δ/L relationship of Polshin and Tokar (1957). For the derivation of $\Delta/L = 0.0003$, L/H is taken as 3. Note that the cracking limits for sagging would be higher (i.e. more tolerant). $\omega = 0.002$ suggests the limit between negligible and slight damage.

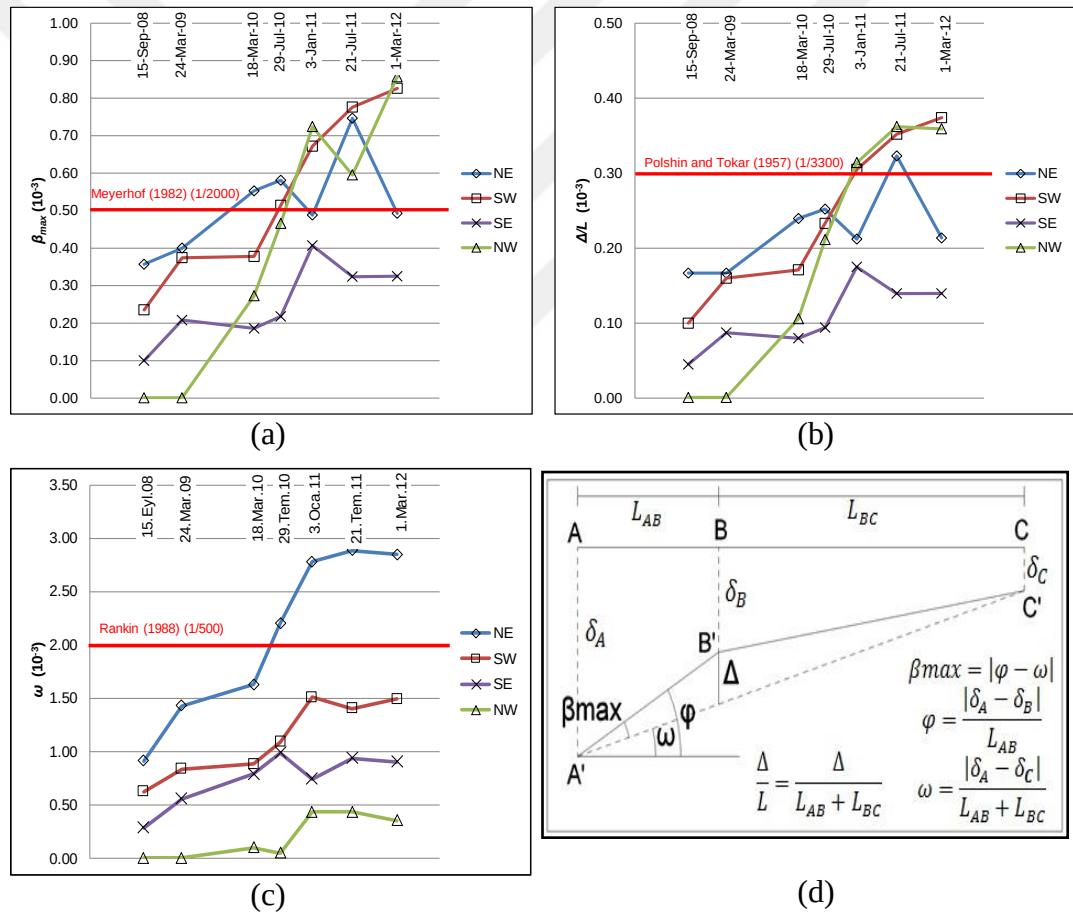


Figure 4.8 : Calculated wall distortions: (a) angular distortion, (b) deflection ratio and (c) rigid rotations.

The β_{max} versus time and Δ/L versus time graphs in Figures 4.8 (a) and 4.8 (b) show that the wall distortions due to WVS (according to values dated 15 September 2008) and curvilinear NPLT (according to values dated on 24 March 2009) excavations are lower than the proposed limit values. Enlargement excavations in CE and SPFV result

in an increase in β_{max} and Δ/L values of the NE and NW walls (see the variation between 24 March 2009 and 18 March 2010). The pilot tunnel excavation in NPFV became very influential between April 2010 and July 2010. In this period, a sharp increase in the distortion values of the most of the walls is observed. Until the completion of the NPFV excavations in January 2011, the wall distortions continue increasing with the exception of the NE wall. In the last two periods (3 January - 21 July 2011 and 21 July 2011 - 1 March 2012), the NE and NW walls experience both increases and decreases in their distortions. Conversely, the distortions of the SW wall monotonically increases in these two periods. The distortions of the SE wall are relatively small. On the other hand, while the NE wall experiences the highest rigid rotation, the NW wall has the lowest value (Figure 4.8 (c)). The calculated maximum rigid rotation is 0.0028 (1/345) for the NE wall and slightly exceeds the limit proposed by Rankin. From 3 January 2011 onwards, rigid rotation values almost level off. At the end of the PFV excavations (January 2011), considerably different rigid rotations are experienced by the parallel NE and SW walls. Presumably, this situation might have resulted in additional shear forces in the connecting SE and NW walls and at the corner connections.

Note that the crack activities given in Figure 4.6 are in general agreement with the wall distortions given in Figure 4.8. For instance, the lowest activity in the C10 crack is in line with the lowest distortions experienced by the SE wall and the variations of the width of crack C3-1 are in line with the increases and decreases in the distortions of the NW wall. However, a direct comparison between crack activities (openings and closings) and variations of calculated wall distortions may not always be straightforward due to:

- Used crack measurement system: The crack width measurements are carried out at only one single location on the crack line, which is selected where the initial crack width is the largest. This kind of a measurement method may easily overlook the detailed crack activities (openings and closings) that can vary along the crack line.
- Distortion calculation method: The pre-existing cracks are local features within the entire wall and their openings and closings partly depend on local mechanisms. On the contrary, the conventional evaluation technique (Figure 4.8) based on the variations of two different distortion measures (angular

distortion and deflection ratio) is not expected to predict such detailed and local mechanisms. Distortion measures are basically calculated to obtain a global idea regarding the settlement response of the wall. Therefore, they are mostly related to the overall response of the wall rather than a localized crack.

- The settlement variation between monitoring points is assumed linear. In reality, this is not perfect linear.
- 3D effects: There might also be out of plane bending behaviour due to three dimensional effect of ground movements and specifications of the structural load-bearing system that will eventually cause partial crack openings or closings. (Such a numerical observation will be presented in the Section 4.5).

4.4.3.2 Analytical evaluation (using the LTSM)

In this section, an analytical evaluation is made using the LTSM for the most distorted wall SW. The input values and calculations are shown in Appendix B. The calculations are based on the measured settlements at the end of July 2011 when NPLT, SPFV and NPFV tunnels were completed (Figures 4.4 and 4.7(b)). Considering Giardina's et al. (2015b) suggestions and based on the percentage area of wall openings to the area of the entire wall (18 %), longitudinal to shear stiffness ratio (E/G) is obtained as 10 (by interpolation). This value corresponds to a flexible structure. The maximum tensile strain in bending and diagonal tension is calculated using equations 2.14 and 2.15 as $\varepsilon_{bmax} = 0.015\%$ and $\varepsilon_{dmax} = 0.030\%$, respectively. ε_{dmax} which is higher is decisive and corresponding damage class is determined as negligible after comparing ε_{dmax} value to the tensile strain ranges given in Table 2.3. According to Burland et al. (1977), the cracks up to 0.1 mm width (almost no visible crack) can be expected for this damage class. Site observations showed that the opening and closing of the pre-existing cracks in this wall are more prominent rather than new visible crack occurrence.

In the current LTSM, the effect of pre-existing cracks is not accounted for. The method assumes the buildings as elastic beams. The effect of structural details such as window and door openings in a load-bearing masonry wall is considered indirectly by setting different longitudinal to shear stiffness ratios (E/G). Therefore, application of finite element method is useful if the effect of structural varieties (i.e. the wall openings) are to be investigated. In sub-section 4.5.1, numerical models for each single wall of this

building with/without pre-existing cracks and with openings are presented. The results show that considering the presence of pre-existing cracks and openings is influential on the response. Furthermore, it is shown that the LTSM might be unconservative because of the strain concentration around the large openings (see Appendix B for the application of the LTSM for this building).

4.5 Proposed Two-Stage Numerical Assessment Approach

The response of the Hoca Pasha Mosque to the complicated spatial ground settlements is examined by 3D finite element modelling (DIANA v10.1, 2016). For this purpose, a two-stage numerical analysis approach is used. In the first stage (in sub-section 4.5.1), the adequacy of the building model with and without pre-existing cracks is primarily verified by using monitored wall settlements. In this way, the issue of setting a realistic soil-structure interaction is temporarily put off. In the second stage, verified building model is analysed using estimated free-field ground movements considering soil-structure interaction.

The geometry of the used building model is shown in Figure 4.9 (a). Recalling that the walls have a thickness of 900 mm and diaphragm actions of the roof is not relevant for the type of loading, the mezzanine floor, the roof, the internal columns and the minaret are not included in this model since their contribution to the overall stiffness is considered as negligible. These sections might have minor damage due to differential settlements. Since this damage will be less critical, the investigation of the response of these sections is out of focus of the current study. The masonry walls are modelled using solid tetrahedron (3 sides, 4 nodes) elements (TE12L). Meshed element size is 250 mm in all dimensions. Bricks and mortar joints are not considered separately in the model and masonry is represented homogenously using linear elastic material properties (the continuum model described previously by Lourenco, 1996) (Table 4.4). Due to the historical character of the mosque, no destructive test on the masonry could be performed to determine the actual mechanical properties. Instead, three different Young's modulus (E_{m1} , E_{m2} and E_{m3}) are adopted. E_{m1} and E_{m2} (350 and 1000 MPa, respectively) are assumed to cover the range of representative values of these typical load bearing masonry walls, constructed in the same era with similar techniques and materials (Ispir and Ilki, 2013). The higher E_{m3} (10000 MPa) is included to distinguish the effect of Young's modulus (Table 4.4).

The localized and already open pre-existing cracks are simulated with vertically aligned linear elastic discrete interfaces. The interface elements are triangular elements (T18IF) which are inserted between two planes in a 3D configuration. They have 3 opposite nodes on each side of the interface and each node has 3 translation degrees of freedom in x , y and z axes. In order to reveal the effect of localized cracks, two extreme conditions are considered: cracks with very low interface stiffness (almost zero) and excluding the crack interfaces from the model (no cracks). For the former, interface stiffness values in the normal direction k_{czi} are calculated theoretically using equation (4.1). That is, the interface stiffness is set as the equivalent stiffness of an fictitious masonry tie of length L_f . The very low interface (almost open) stiffness is obtained by considering L_f , arbitrarily, as 10^{10} mm. The crack interface stiffness in the tangential x and y directions (k_{cxi} and k_{cyi} , respectively) is taken as relatively small, although not extremely small, ($E_{mi} \times 10^3$ in kN/m^3) to take into account small frictional effects between crack faces and to avoid large unrealistic out-of-plane drifts of the wall portions between the cracks (Table 4.4).

$$k_{czi} = \frac{E_{mi}}{L_f} \times 10^6 (\text{kN/m}^3) (\approx 0 \text{ for any } E_{mi}) \quad (4.1)$$

Table 4.4 : Assigned material parameters.

Property			
Masonry wall	Young's modulus (MPa)	E_{m1}	350
		E_{m2}	1000
		E_{m3}	10000
	Poisson ratio	ν	0.2
	Unit weight (kg/m^3)	γ_m	1800
Localized cracks	Interface stiffness in normal direction (kN/m^3)	k_{czi}	$(E_{mi}/L_f) \times 10^6 \cong 0$
	Interface stiffness in tangential directions (kN/m^3)	$k_{cxi} = k_{cyi}$	$E_{mi} \times 10^3$
Bedding	Interface stiffness in normal direction (kN/m^3)	k_{bz1}	5000
		k_{bz2}	20000
		k_{bz3}	40000
		k_{bz4}	80000

Monitored wall settlements (considered in the first stage analyses) and estimated free-field ground settlements (used in the second stage analyses) are imposed to the bedding interfaces at the bottom surfaces of the solid walls. The same element type (T18IF) as

used to model the pre-existing cracks is used for the bedding. Figure 4.9 (b) schematically explains the relationship between applied and resultant displacements on the each side of the bedding interfaces. In this figure, $\delta_v(x,y)$ is the applied vertical prescribed deformation and $\delta_{wall}(x,y)$ is the resultant displacement at the bottom of the walls of the structure. For the increasing values of k_{bz} the modulus of subgrade reaction (referred to as bedding stiffness hereafter) δ_{wall} approaches to δ_v , i.e. for $k_{bz} \rightarrow \infty$, $\delta_{wall} = \delta_v$. The latter is equivalent to excluding the bedding interface from the model. Lower values of k_{bz} (corresponds to a softer soil) will lead to relative displacements in the normal direction of the bedding interface, $\delta_v - \delta_{wall}$. The effect of using a nonlinear soil model in the analyses has been investigated by the previous researchers (Burd et al., 2000 & Fagnoli et al., 2015). It was shown that more realistic simulations are possible by considering small-strain nonlinearity and stiffness reduction with increasing deformation in the soil model. However, in this study, a linear elastic material model with a constant bedding stiffness is used. Although the bedding stiffness is not a specific material property for soil, Bowles (1996) suggested that guide values given in Table 4.5 can be used to estimate the correct order of this quantity. Utilizing Bowles's guide values, four different bedding stiffness values varying in a broad range are considered in the parametric analyses (5000, 20000, 40000 and 80000 kN/m³) (Table 4.4). Among these, $k_{bz3} = 40000$ kN/m³ is determined as the reference value considering the depth and type of underlying soil layers in the region. The bedding stiffness in transverse direction is assumed close to zero. Note that the bedding interface does not contribute to the flexural stiffness of the wall; its main function will be the monitoring of the relative displacement distribution underneath the walls and to simulate the effect of soil layers underneath the walls.

Table 4.5 : Guide values proposed by Bowles (1996) for bedding stiffness.

Soil type	k_{bz} (kN/m ³)
Loose sand	4800-16000
Medium dense sand	9600-80000
Dense sand	64000-128000
Clayey medium dense sand	32000-80000
Silty medium dense sand	24000-48000
Clayey soil	
$q_a \leq 200$ kPa	12000-24000
$200 < q_a \leq 800$ kPa	24000-48000
$q_a > 800$ kPa	>48000

Note: q_a is the allowable bearing pressure.

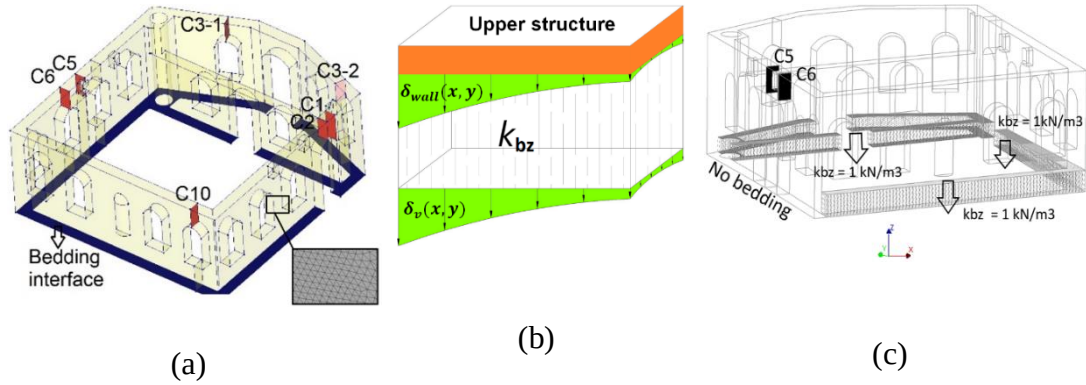


Figure 4.9 : (a) Mosque model geometry (b) Interface model and (c) interface configuration for the isolated SW wall.

4.5.1 The first stage analysis results for building model verification by using measured settlements

In order to verify the building model, this section presents preliminary analysis results. These analysis results are based on two temporary simplifications of the numerical model:

1. The critical issue of defining the settlement fields and a realistic bedding stiffness are avoided, by using measured wall settlements as displacement loads at the bottom of the building.
2. The response of each load-bearing wall to the displacements imposed at the bottom of that wall is analysed for each wall separately, by ignoring the loading effects from the three remaining walls, but by including the flange effects (stiffness contribution) of other connecting walls.

As an example, Figure 4.9 (c) presents the modified analysis model for the SW wall. No bedding interface is assigned at the bottom of this wall: the applied prescribed deformations (i.e. the measured settlements) will be transferred directly to the wall. The analyses performed in this section are based on the measured settlements at the end of July 2011 when NPLT, SPFV and NPFV tunnels were completed (Figures 4.4 and 4.7 (b)). The measured settlements are distributed along the wall length based on the assumption that the settlements are varying linearly between the monitoring points SB8, SB19 and SB18. On the other hand, an extremely soft bedding interface stiffness (1 kN/m^3) is assigned to the bottom of the remaining walls. In this way, the rest of the structure still exerts a stiffness influence on wall SW. The Young's modulus E_{m1} and E_{m3} for the masonry walls are used in successive analyses. The SW wall includes the

C5 and C6 cracks (defined with a very low normal stiffness). In a variation study, these cracks are excluded. The cracks on the other walls are always excluded from the model.

Figures 4.10 and 4.11 show the deformation and crack opening results obtained for the SW wall without and with crack interfaces (C5 and C6), respectively. In the first case, it is seen that the major principal tensile strains (ε_p) concentrate around the small niche at the upper side of the wall as well as the bottom parts of the central windows. The maximum principal tensile strain (ε_{pmax}) reaches about 0.080%. This value exceeds slightly the tensile strain range (0.038% - 0.060%) that was previously reported by Burhouse (1969) as the onset of visible cracking in the brick masonry walls. The presence of the C5 and C6 cracks in the second case results in a remarkable change in ε_p distribution. It is seen that the C5 and C6 cracks reduce the strain levels in their vicinity, however they result in an increase in the strain levels at the bottom parts of the central windows. The maximum principal tensile strain reaches about 0.1%. In both cases, the damage class of SW wall is assessed slight damage based on the comparison made between ε_{pmax} and the suggested limiting tensile strain intervals of Boscardin and Cording (1989) (see Table 2.3 in Section 2).

The variation of the relative crack displacements (δ_{cz}) along the height of the C5 and C6 cracks are shown in Figures 4.11 (b) and (c), respectively. Crack C6 is opening more uniformly and widely, whereas C5 widens more at the upper part and relatively less. A direct comparison of numerical and monitored crack openings is not straightforward due to aforementioned restrictions. Instead, making comparisons based on the summed crack openings of the two cracks makes sense. The analysis yields that the total crack opening of C5 and C6 (C5 + C6) ranges from 7 to 9.5 mm from the bottom to the top of the cracks. The sum of the monitored crack openings (C5+C6) (at the end of July 2011 in Figure 4.6) is 5.3 mm.

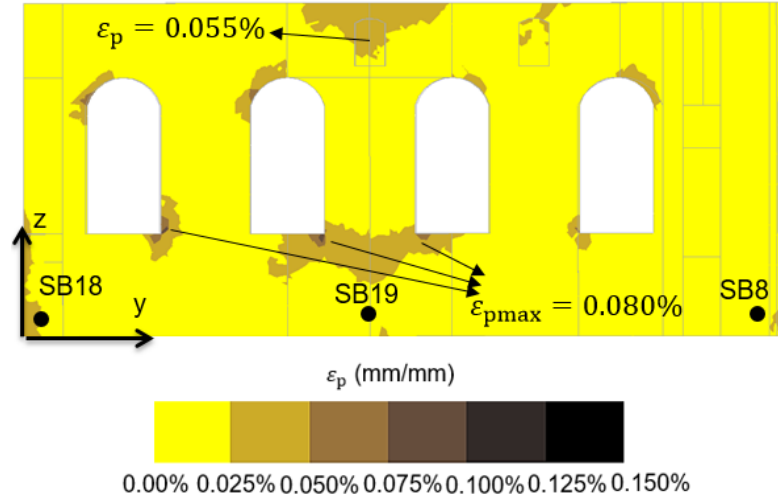


Figure 4.10 : ε_p distribution in SW without C5 and C6 crack interfaces ($E_m = 350$ MPa).

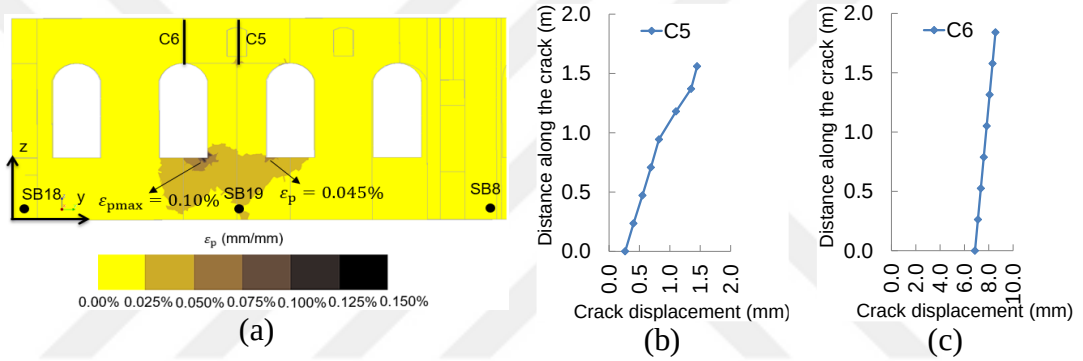


Figure 4.11 : (a) ε_p distribution in SW wall with crack interfaces and relative crack displacements of (a) C5 and (b) C6 cracks ($E_m = 350$ MPa).

Similar modelling and analysis procedures are executed for the NE, NW and SE walls. The results obtained from these analyses are summarized in Table 4.6. As seen, the magnitude of shown ε_{pmax} values are relatively low as in the case of SW wall. This would result in negligible to slight damages for these walls (Table 2.3).

An in-plane rotation is observed for the wall portion between the C1 and C2 cracks in NE wall (Figure 4.12 (a)). Due to this rotation, while the C1 closes at the upper part C2 widens and the other way around at the bottom part as observed from the outside of the mosque. For these crack interfaces, the overall relative crack displacement (δ_{cz}) from bottom to upper tip varies between -0.33 mm (closing) and 4.71 mm (opening). On the other hand, a diagonal out-of-plane rotation is observed for the C3-2 crack in the NW wall (Figure 4.12 (b)). Finally, the SE wall experiences the lowest distortion compared to other walls in which the C10 crack tends to close.

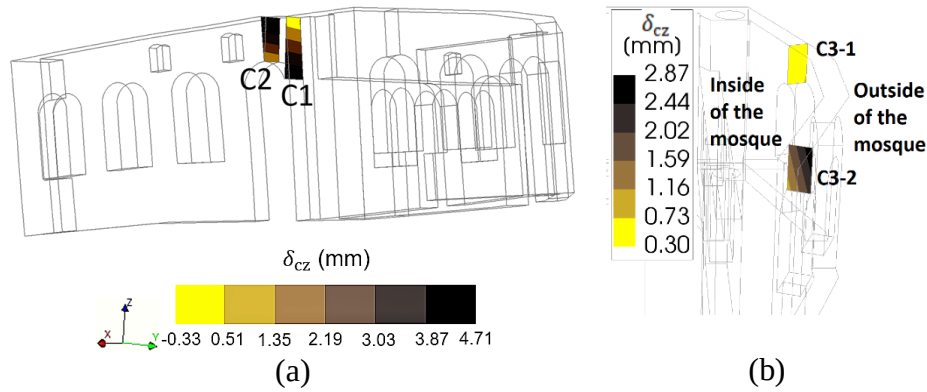


Figure 4.12 : Rotation of crack interfaces from (a) perspective (C1&C2) and (b) side view (C3-2).

Table 4.6: Numerical results for NE, NW and SE walls (with and without pre-existing cracks).

Wall	Max. principal strain (without cracks) ($E_m = 350$ MPa)	Max. principal strain (with cracks) ($E_m = 350$ MPa)	Remarks for relative crack displacements	Estimated damage class according to Table 2.3
NE	$\epsilon_{pmax} = 0.060\%$ Near to upper and lower section of the central part of the wall	$\epsilon_{pmax} = 0.055\%$ Near to upper and lower section of the central part of the wall	Figure 3.12(a)	Very slight damage in both cases
NW	$\epsilon_{pmax} = 0.075\%$ Observed at several sections near to window and wall corners	$\epsilon_{pmax} = 0.075\%$ Observed at several sections near to window and wall corners	Figure 3.12(b)	Slight damage in both cases
SE	$\epsilon_{pmax} = 0.025\%$	$\epsilon_{pmax} = 0.025\%$	C10 uniformly closes	Negligible damage in both cases

Note that varying the Young's modulus of the masonry from 350 MPa (E_{m1}) to 10000 MPa (E_{m3}) results in insignificant changes in the magnitudes of the principal strains and crack openings. As seen in Table 4.6, the obtained crack activities and determined damage classes based on the tensile strain levels are in line with the actual monitoring results and in-situ building observations. Recall that, during the excavation and construction works, while the pre-existing cracks were active a few new cracks of insignificant extent have occurred. The agreement between the results of the first stage analyses and reality shows the suitability of the building model and choice of the range of the elastic material parameters.

4.5.2 Second stage parametric analyses performed by using estimated free-field settlements

In this sub-section, the settlement response of the mosque will be examined using parametric analyses based on generated settlement fields. The rational of this kind of analyses compared to the previous ones, in which measured settlements are used as input, is to offer an opportunity to examine the crack activities and overall response without having the knowledge of the building settlements. Furthermore, the effect of different masonry and bedding properties and modelling configurations (i.e. with or without crack and bedding interfaces) are investigated in this section.

Due to aforementioned limitations of in-situ free-field ground settlement data, 3D settlement fields have to be estimated. For the shaft excavation, WVS, 3D settlement field (Figure 4.13 (b)) is produced based on empirical 2D free-field settlement profile curve (Figure 4.13 (a)). For NPLT, SPFV and NPFV tunnels, 3D settlement fields (Figures 4.13 (c), (d) and (e)) are based on reported 2D finite element models (Figure 4.13 (a)). Each settlement field is formed as a vertical settlement displacement matrix and applied as a prescribed deformation load to the interfaces at the bottom of the walls. The combined settlement field (Figure 4.13 (f)) is the superposition of all of the matrices formed for each separate excavation (WVS, NPLT, SPFV and NPFV). Details of the sub-studies to estimate 3D settlement fields for WVS and tunnels are explained in the following sub-sections 4.5.2.1 and 4.5.2.2.

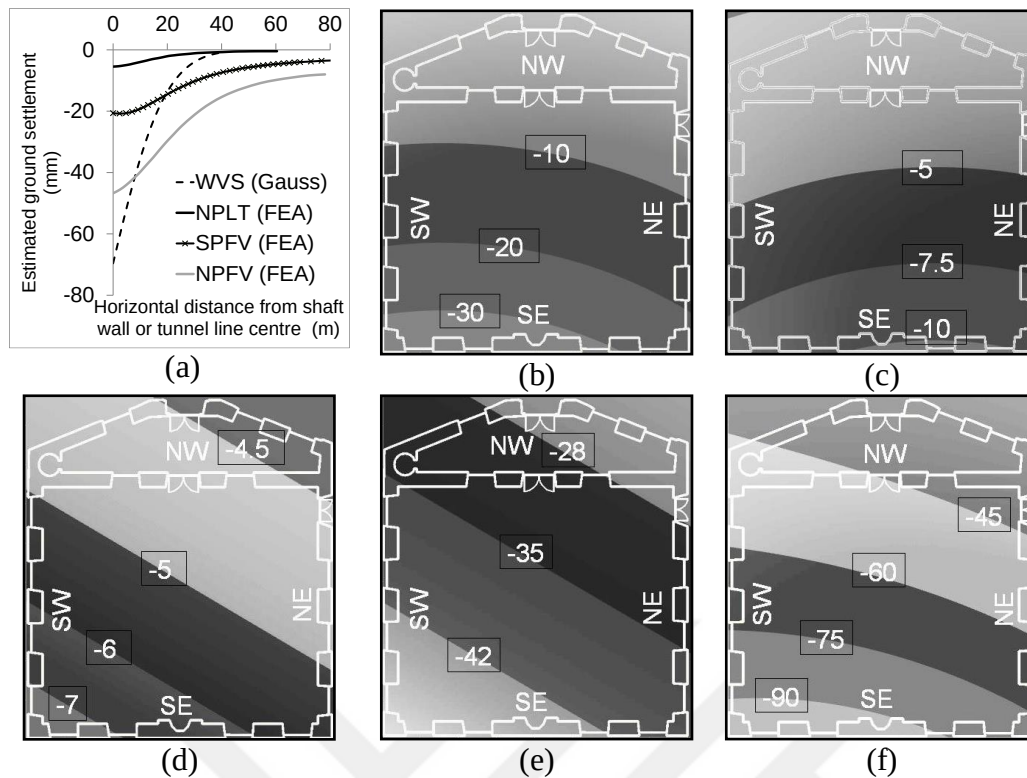


Figure 4.13 : (a) Reported or derived 2D free-field ground settlement estimations which are used to produce settlement fields and 3D settlement field contours generated for (b) WVS, (c) NPLT, (d) SPFV, (e) NPFV and (f) combination of WVS, NPLT, SPFV and NPFV excavations.

4.5.2.1 Derivation of the 3D settlement field for WVS from the calculated empirical ground settlements

For the estimation of free-field ground settlements and the derivation of corresponding settlement field due to the shaft excavation, the study of Moorman (2004), in which 530 case histories of retaining walls and ground movements due to deep excavations are analysed, is used. Moorman shows that the movements of the shaft wall and the surrounding ground seem to be largely independent of the stiffness of the retaining wall system. According to the author, once a sufficient stiffness is provided, the movements are governed by other relevant factors. Thus, an additional increase of the system stiffness does not lead to a corresponding decrease of movements. Similar conclusions were obtained by Clough and O'Rourke (1990) and Long (2001). Considering this, the mixed type wall construction of the current shaft can be considered as single type wall and empirical settlement relationships proposed for uniform retaining wall constructions can be used for the entire depth of the shaft ($H_e = 58$ m). Moorman (2004) also shows that the maximum ground settlements (δ_{vm}) behind an excavation wall can vary in a wide range. For relatively stiff clays

(undrained shear strength $c_u \geq 0.075$ MPa), he observed that the maximum ground settlement (δ_{vm}) to excavation depth (H_e) ratio takes a value between 0% (no ground settlement) and 0.90%. The average δ_{vm}/H_e ratio is reported as 0.18%. A similar range (0.00-0.20%) for the average value of δ_{vm}/H_e was also previously proposed by Clough and O'Rourke (1990) and Long (2001).

In the current study, considering the magnitude of measured settlements, an average value of $\delta_{vm}/H_e = 0.12\%$ which is in the proposed range can be assumed. In this case, δ_{vm} is calculated as 69.6 mm for $H_e = 58$ m. In order to calculate the corresponding ground settlement profile for $\delta_{vm}/H_e = 0.12\%$, Peck's (1969) Gaussian formula (equation 4.2) can be used as introduced in the study of Lee et al. (2007). This curve was originally suggested to estimate free-field ground settlement profiles due to tunnelling. Afterwards, Lee et al. (2007) used this function to predict the excavation-induced ground settlement profile behind an excavation wall assuming that the wall stands at the inflection point of the Gauss curve. In equation 4.2, δ_v is the ground settlement at any distance r from the shaft wall and W shows the settlement trough width proposed by Caspe (1966). W can be calculated using equation 4.3. Therein, H_i is settlement influence depth below the excavation level and φ is the friction angle of the soil. H_i is calculated according to equation 4.4 for soils with $\varphi > 0$. φ is assumed to be 30° in the current case. B shows the excavation width and can be taken as 24.6 m as the average diameter of the elliptic shaft in the current study.

$$\delta_v = \delta_{vm} \cdot e^{\left[0.5 - 0.5\left(1 + \frac{2r}{W}\right)^2\right]} \quad (4.2)$$

$$W = (H_e + H_i) \tan\left(45 - \frac{\varphi}{2}\right) \quad (4.3)$$

$$H_i = 0.5B \tan\left(45 - \frac{\varphi}{2}\right) \quad (4.4)$$

Figure 4.13 (b) shows the contours of the settlement field which is produced for the shaft excavation based on the Peck's (1969) Gaussian curve (Figure 4.13 (a)). Note that the 3D settlement field was constructed by applying the 2D Gaussian curves in radial directions of the elliptical shape of the shaft starting from the position of the shaft wall.

4.5.2.2 Derivation of the 3D settlement fields for NPLT, SPFV and NPFV tunnels from the 2D numerical results

Free-field ground settlements due to tunnel excavations were obtained from the numerical analysis results reported by the contractor company (Figure 4.13 (a)) (Prediction of Ground Surface Settlement Report, 2011). In these analyses, 2D plane strain conditions were considered. Man-made fill (I) and clayey silty sand (II) layers were modelled using an elasto-plastic Mohr-Coulomb model. Besides the elastic properties, anisotropic damage parameters were used to reduce the shear modulus of the greywacke (III). Damage parameters were determined through back analyses based on the monitored crown and spring line settlements of the concerned tunnels. Figure 4.13 (c), (d), and (e) show the contours of the settlement fields which are produced based on the numerical simulations (Figure 4.13 (a)). As seen, the curvilinear NPLT excavation yields curved settlement contours and the SPFV and NPFV form skewed line contours corresponding to the geometry and position of these tunnels. In line with the monitoring results of the mosque (Figure 4.4), the settlement contours resulting from the NPFV excavation indicate much higher settlements in comparison with the NPLT and SPFV excavations. Figure 4.13 (f) shows the combination of all settlement contours resulting from the WVS shaft and NPLT, SPFV and NPFV tunnels. Note that this combined settlement field corresponds to a site settlement stage when the NPLT, SPFV and NPFV tunnels were completed (at the end of July 2011). A sagging due to combination of the settlement contours of the WVS and NPLT can be noted for the SE wall. The other walls are hogging. This situation is also in line with the monitored responses of these walls. However, two discrepancies between estimated and actual settlements should be highlighted and the probable reasons should be discussed:

- Although the estimated settlements are increasing towards the SE-SW corner (SB18) due to closeness of this corner to the WVS, SPFV and NPFV excavations (see Figure 4.2 (a) and Figures 4.13 (b), (d) and (e)), a more severe settlement was measured in reality for the SE-NE corner (SB14). The reason of reading higher settlement from SB14 monitoring bolt might be due to a possible local discontinuity and stiffness difference in the relatively thick and loose man-made fill. The region has been a trade and residential centre over the centuries and many archaeological relics including ancient building parts and wells have been discovered (Figure 4.14) (Iwano et al., 2013).



Figure 4.14 : Archaeological building remains in a shaft excavation in the region.

- Since the influence of other neighbouring tunnels (SPF, CE and ISL) cannot be considered in the generation of the combined settlement field, the magnitudes of the estimated combined settlements is slightly lower than the measured values. Although these excavations contributed to an extent to the settlement of the mosque, they cannot be included in the settlement field calculations due to geometrical difficulties. In order to consider the effect of SPF and CE, interaction between SPF&CE and shaft wall should be investigated by further analyses. Similarly, accounting for the effect of ISL excavation that have a different alignment (sloping upward to the ground surface) would require physical modelling of this excavation in 3D.

Despite these limitations, the generated settlement fields are believed to reflect the basics of the ground movements near the Hoca Pasha Mosque.

4.5.2.3 Second stage analysis results

The second stage analyses are performed using the combined 3D settlement field shown in Figure 4.13 (f) (Table 4.7). The Analyses 1 (reference), 2 and 3 aim to investigate the effect of the Young's modulus of the masonry. Analyses 1, 4, 5 and 6 investigate the effect of the bedding stiffness using a constant Young's modulus of the masonry. Analyses 7 and 8 study the effect of the leaving out the pre-existing crack and bedding interfaces, respectively. In analysis 9 both interfaces have been left out.

Note that, all these are linear analyses, based on the following assumptions:

- The bedding interface will remain under compression when considering all loads: any potential separation between soil and building is surpassed.
- The crack interfaces will remain open: the need for an increased stiffness value in case of a crack closure is left out.

Using constant stiffness properties for these interfaces is then justified. It has been verified that with the inclusion of the building weight all bedding interfaces will indeed remain under compression for all analyses. The crack interfaces represent cracks with an initial crack opening in the range of 5 to 10 mm. No detailed information about the exact openings is available. For some of the cracks and for some analyses a maximum closure is observed of 10 mm for a part of the crack, which thus suggests a full closure of cracks at some locations and would demand for an increase of interface stiffness. However, this effect has been left out in the analyses, since the occurrence of possible crack closures is very limited and because of the limited available data on initial cracks openings. An additional advantage of linear analyses is that superposition allows to analyse only the effect of the settlement loads which will facilitate the interpretation of the results.

Table 4.7 : The features of the parametric analyses.

Property	Analysis 1 (reference)	An. 2	An. 3	An. 4	An. 5	An. 6	An. 7	An. 8	An. 9
Young's modulus of the masonry (E_{mi}) (MPa)	350	1000	10000	350	350	350	350	350	350
Bedding stiffness (k_{bz}) (kN/m ³)	40000	40000	40000	5000	20000	80000	40000	No bedding	No bedding
Presence of pre-existing cracks	Yes	Yes	Yes	Yes	Yes	Yes	No	Yes	No
Used settlement field	Combined (Figure 4.13(f))								

The contours of the principal tensile strain (ε_p) and relative displacements of the crack and bedding interfaces (δ_{cz}) obtained through the Analyses 1-9 are presented in Figures 4.15-4.17. In addition, the ranges of the variation of the openings and closings throughout the concerned crack and bedding interfaces are shown comparatively in the Appendix C in Figure C.1 for each analysis. Note that, in Figure C.1, while negative values show closings positive values show either openings in the crack interfaces or partial unloading of bedding interface from one compressed state of the soil to another. The values illustrated by the red circles are the in-situ crack measurements at the end of July 2011.

The results of Analyses 1, 2 and 3 show that an increasing structural stiffness through the increase of the Young's modulus of the masonry walls affects the conformity of the walls to the applied prescribed deformations. The more homogenous distribution of the bedding displacements (relative displacements of the bedding interface are being close to zero) in Analysis 1 indicates that the structure, which has the lowest stiffness, mostly conforms to the applied prescribed deformations (Figures 4.15 and C.1). This situation eventually results in higher principal tensile strains within the structure. Increasing structural stiffness results in increased relative displacements in the bedding interface (Figures 4.15 and C.1). The magnitude of the maximum principal tensile strain decreases and the structure tends to remain undeformed. This result is completely in line with the findings of previous studies of Potts and Addenbrooke (1997) and Son and Cording (2005). They established a relationship between soil/structure stiffness ratio and potential building distortions and observed that as this ratio decreases, the building distortions decrease.

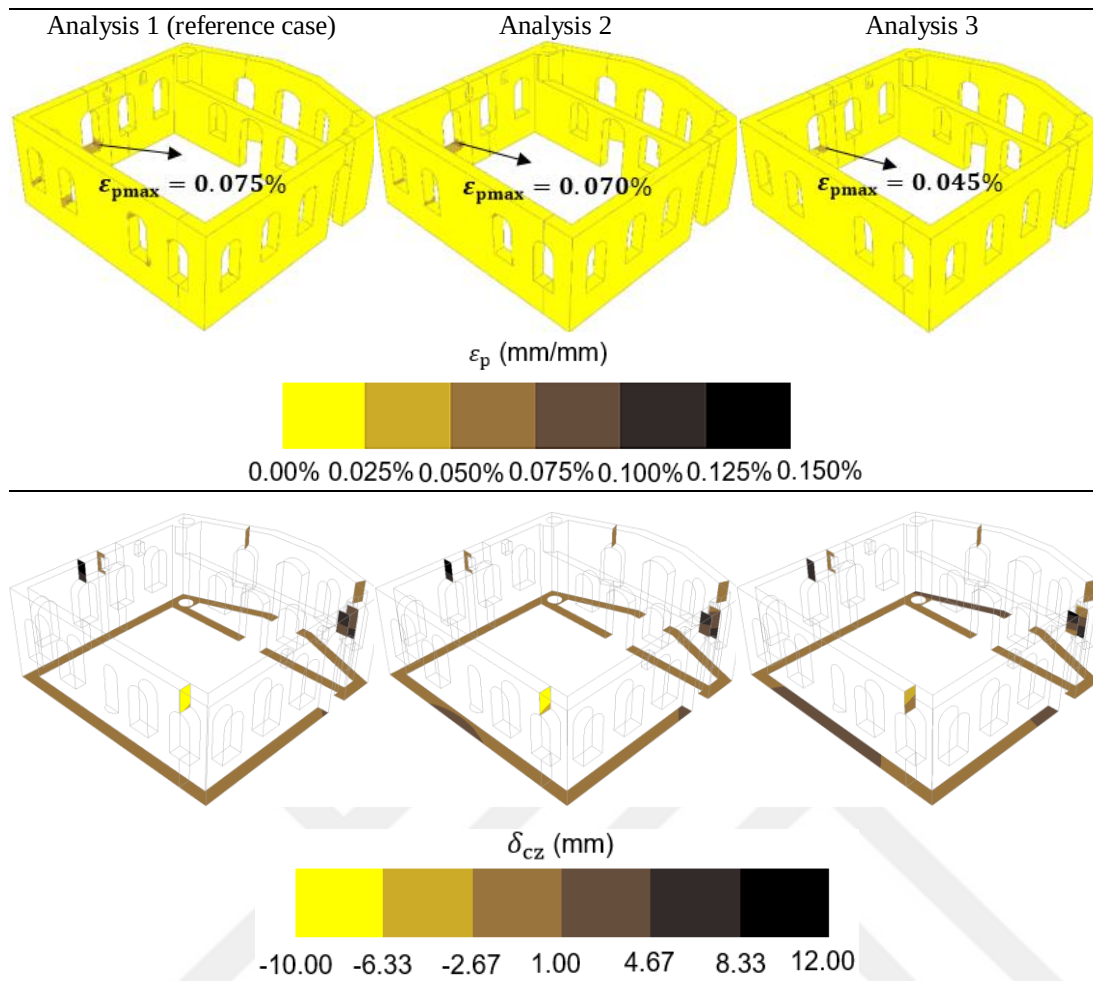


Figure 4.15 : The results of the Analyses 1, 2 and 3 (Variation of the masonry stiffness).

As the structural stiffness increases, C1 tends to widen at its bottom part and close at its upper part in a more pronounced manner. The C2 acts the other way around due to rotation of the wall portion between C1 and C2 crack interfaces as observed in the first stage analysis of NE wall in sub-section 4.5.1. The summed magnitude of the relative displacements of C5 and C6 crack interfaces decreases slightly. Cracks C3-1 and C3-2 have the lowest relative displacement regardless increasing stiffness. As the Young's modulus increases, C10 changes to a partial opening from a closure (Figures 4.15 and C.1).

Increase of the soil stiffness is simulated with the increase of bedding stiffness. As the bedding stiffness is increased from k_{bz1} to k_{bz4} (through the Analyses 4, 5, 1 and 6, respectively) a remarkable increase of principal tensile strain in the walls is observed (Figures 4.15 and 4.16). This is because higher bedding stiffness yields higher stress in comparison to a soft bedding subjected to same magnitude of prescribed settlement

deformation. Recall that increasing soil/structure stiffness ratio results in increases of the structural distortions. On the other hand, a stiffer bedding interface leads to a more uniform relative displacement distribution which almost equals to zero in magnitude (see Analysis 6). As the bedding stiffness increases, the relative displacements of the localized cracks change insignificantly. Analysis 8 can be considered as the extreme case in terms of the bedding stiffness ($k_{bz} \rightarrow \infty$). For this analysis, while an increase in the tensile principal strain values is observed, the relative displacements of the localized cracks are similar to those obtained for other bedding stiffness values.

Finally, comparing the results of Analyses 1 & 7 and 8 & 9 shows that the presence of pre-existing cracks reduces the principal tensile strains in the close vicinity of the cracks, but results in an increase in other neighbouring sections (Figure 3.17). This result was also observed in the previous sub-section 4.5.1 in which each wall was examined separately. Since the model used in Analysis 9 does not include crack interfaces, a concentration of principal tensile strain appears at the upper sections of the SW and NE walls. Although principal tensile strains reach the highest magnitude in Analysis 8 and 9, the maximum principal tensile strain does usually not exceed 0.14 % which corresponds to slight damage according to Table 2.3. This result is also in line with the results found in previous section and site observations.

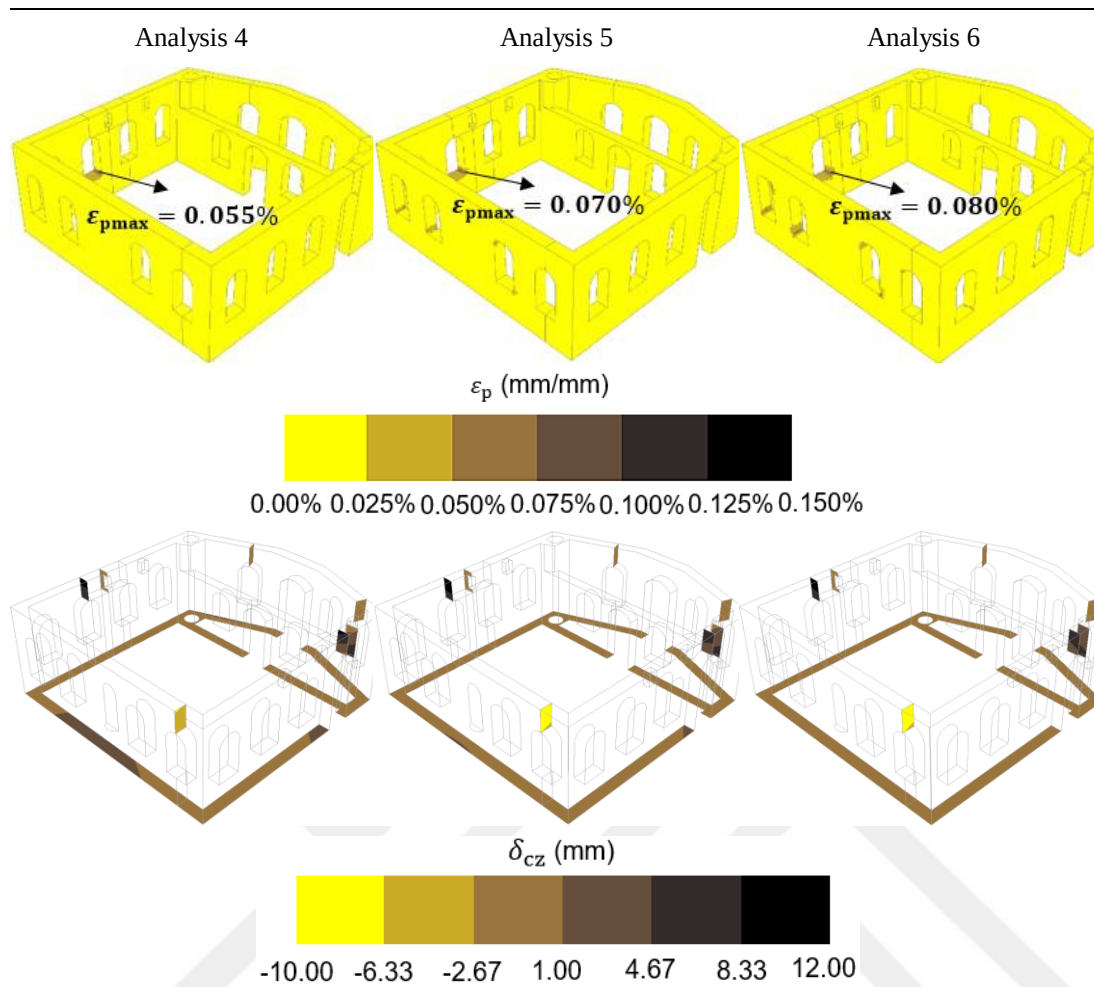


Figure 4.16 : The results of the Analyses 4, 5 and 6 (Variation of the bedding stiffness).

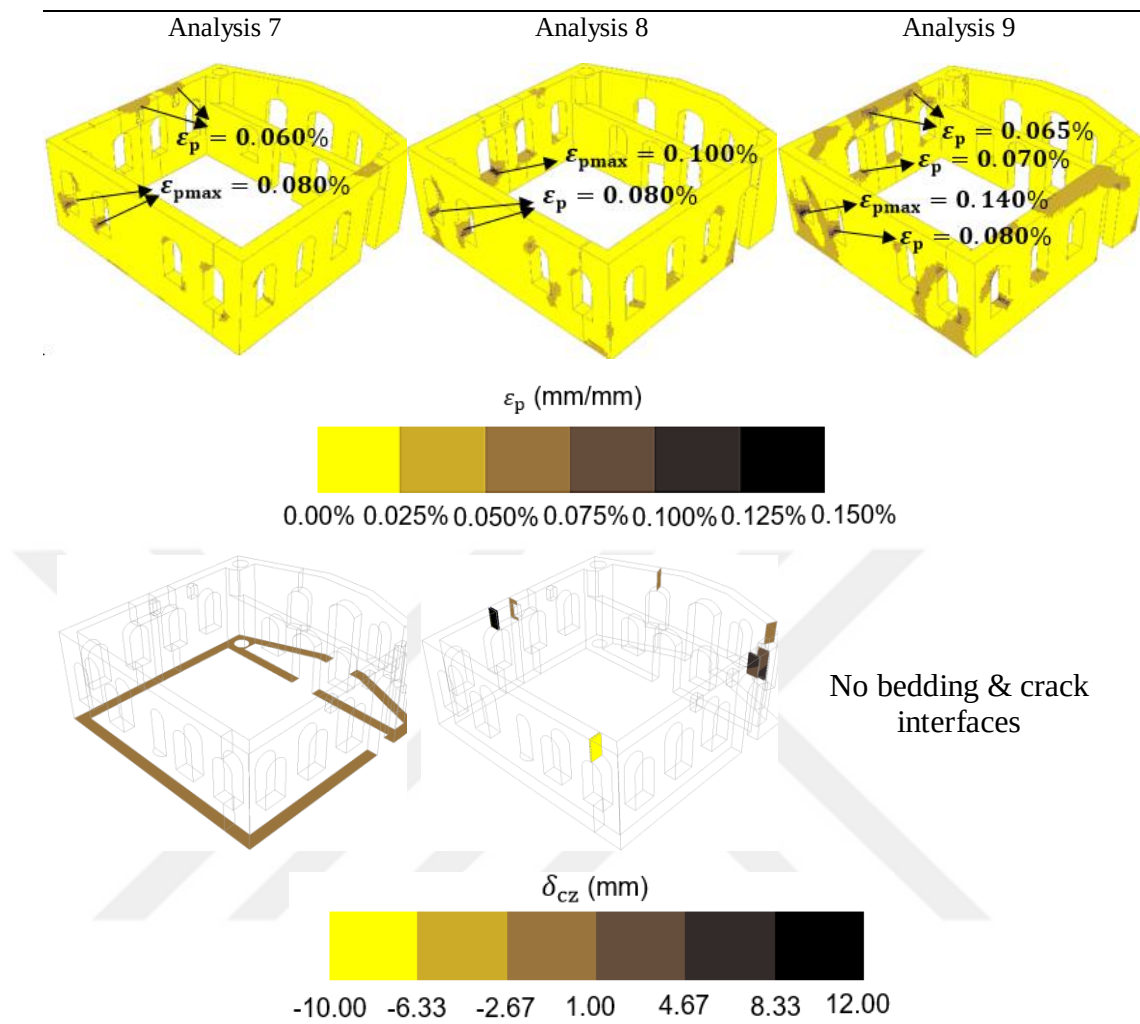


Figure 4.17 : The results of the Analyses 7, 8 and 9 (Variations of pre-existing cracks and bedding).



5. CONCLUSIONS AND RECOMMENDATIONS

In recent years, the number of tunneling and deep excavation projects in Turkey and all over the world has increased. The risk of building damage that will arise due to these activities, which are mostly carried out in urbanized areas, needs to be correctly determined. The LTSM is one of the methods used frequently to determine the building damage and accordingly risk level due to excavation activities. However, the practical application of this method has raised some important questions regarding the simplicity and basic assumptions adopted in this method.

In Chapter 2, an extensive and comparative literature review was first presented and reported advantages and disadvantages of the LTSM were clearly addressed. It was shown that the LTSM as a practical tool is useful in the preliminary assessment stages of the buildings as long as its limitations are considered. However, the findings of the existing experimental and numerical studies showed that the LTSM might be substantially deficient to predict real building damages due to its simplicity.

In Chapter 3, the fictitious deep beam equations proposed by Burland and Wroth (1974) were re-expressed as a transfer function ($\Delta/L = C\varepsilon_{crit}$), and the interpretation of the $\Delta/L - L/H$ relationship was simplified by focusing on the coefficient 'C'. The original deep beam equations were also modified to include the effect of the average horizontal strain. Furthermore, an evaluation of the simply modified and current deep beam equations was performed using the reported data and observed damage classes of case studies available in the literature. The results showed that the observed damage and measured crack widths are more or less compatible with the limiting tensile strain boundaries. It was shown that for the cases in which moderate or higher damage was observed, the original deep beam equations tend to underestimate the resultant damage class. Although masonry buildings with large openings and open frame structures have an increased flexibility in response to ground movements, due to possible damage localizations around openings and beam-column joints, it was shown that the LTSM could make underestimated damage predictions for these buildings.

In Chapter 4, the settlement response of the historic Hoca Pasha Mosque to a nearby network of shaft and tunnel excavations was numerically examined. The LSTM underestimated the damage class of this masonry structure. The problem had a series of challenges due to uncertainties regarding mostly the excavation activities, soil properties, building material and pre-existing damages. These typical difficulties are inherently encountered in such case studies. In order to put off some of these uncertainties a two-stage analysis approach was adopted. First the 3D structural model (with and without existing cracks) was verified in a way that each façade wall is loaded separately using the measured settlements and taking into account flange effects (stiffness contribution) of other connecting walls. This stage to a large extent reduced modelling uncertainties. In the second stage, the effects of other parameters i.e. the building stiffness, bedding stiffness and combined settlement field due to sequential excavation works were investigated. This naturally increased the modelling uncertainties, but was closer to the practice of *predicting* settlement responses.

The following conclusions could be derived from the reproduced response of the mosque:

- Using a two-stage analysis approach, the actual response of the building could be satisfactorily represented. Despite the serious simplifications in modelling, the overall settlements and crack activities could be simulated realistically. Furthermore, the results of the proposed two-stage numerical analysis approach showed that 3D analysis of an isolated single wall of a building can predict the final damage as sufficient as 3D whole building analyses can do. This is a promising result that can lead to an intermediate analysis technique that is more precise than the LSTM but is free of the computational burden of complex 3D complete building finite element analyses. At this point, the advantage of the use of a semi-coupled bedding interface which enables the consideration of the soil-structure interaction without modelling the soil in 3D should also be highlighted. However, in order to put forward an intermediate analysis technique based on the observed merits of the two-stage numerical approach, there is need for more number of analyses including different building and excavation configurations. It is known that researchers from Oxford University were working on this subject at the time of writing this thesis.

- In accordance with the site observations, analysis results showed that the tensile strains in the walls are usually of relatively limited magnitude: a few new cracks of insignificant extent occurred due to settlements.
- Including existing cracks into modelling through the discrete interfaces reduced tensile strains in the close vicinity but resulted in an increase in the tensile strains of other neighbouring sections. While the activity of the existing cracks was considerably influenced by the increase of the Young's modulus of masonry, the variation of the bedding stiffness in the considered range had a limited effect.
- Excluding both bedding and existing crack interfaces during modelling resulted in the most severe case in terms of tensile strain level and tensile strain distribution.

Besides the examination of the specific case of historic Hoca Pasha Mosque, the performed parametric analyses supported by the sub-studies regarding the determination of 3D settlement fields also provide useful outputs to reach more generalizable results. The obtained results confirm the previous findings in the literature:

- Increased structural stiffness through the increase of the Young's modulus of the masonry walls reduced the conformity of the structure to the applied prescribed settlements and led to lower tensile strains.
- Increase of the soil stiffness is simulated with the increase of the bedding stiffness. The higher the bedding stiffness is the more the structure conforms to the applied prescribed settlements. This eventually resulted in increased tensile strains. As an extreme condition, excluding bedding interface corresponded to a case with infinite bedding stiffness.

Based on the findings of the review and simulation studies, the following studies are planned to be conducted in the future:

- The need for large scale laboratory tests: Large scale wall or frame tests can answer many questions regarding the mechanics of the building response to differential ground movements. The effect of stress redistribution, plasticity (or cracking), building weight, structural boundary conditions and soil-structure interaction, orthogonal walls, progressive nature of the ground movements, etc. can be investigated.

- Proposing a new damage classification system: The current damage classification system adopted in the LTSM is based on observations and early experimentations carried out for certain types of masonry wall materials (mostly brick). In contrast, this classification method is used all over the world without paying sufficient attention to masonry wall composition and material. Therefore, the validity of the current damage classification for different masonry wall materials (stone, etc.), compositions, and joint types is questionable. Further studies that consider these variables are needed.
- A numerical study for finer adjustment of E/G : In order to increase the accuracy of the current LTSM, a series of numerical analyses can be performed for the masonry and frame building walls. The walls with varying material properties and in different geometries are numerically analysed and equivalent values of longitudinal and vertical (shear) stiffness can be derived from the global response of these walls. Afterwards, these values can be implemented in the current LTSM.
- Detecting the building vulnerability prior to an excavation activity: A new vulnerability index can be suggested for masonry and frame buildings in Turkey using the existing data bases of completed or on-going tunneling projects. Such an index would offer a rapid assessment of building susceptibility in the preliminary assessment level.
- Suggesting a special monitoring scheme: In order to improve and/or validate the current numerical and analytical prediction techniques, suggesting a robust monitoring scheme for overall displacements of ground, buildings and underground structures is a need. In this way, required data can be collected in a more systematic way.

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APPENDICES

APPENDIX A: The application of the analytical method (LTSM) to predict the damage class of the facade wall tested in Giardina et al. (2012).

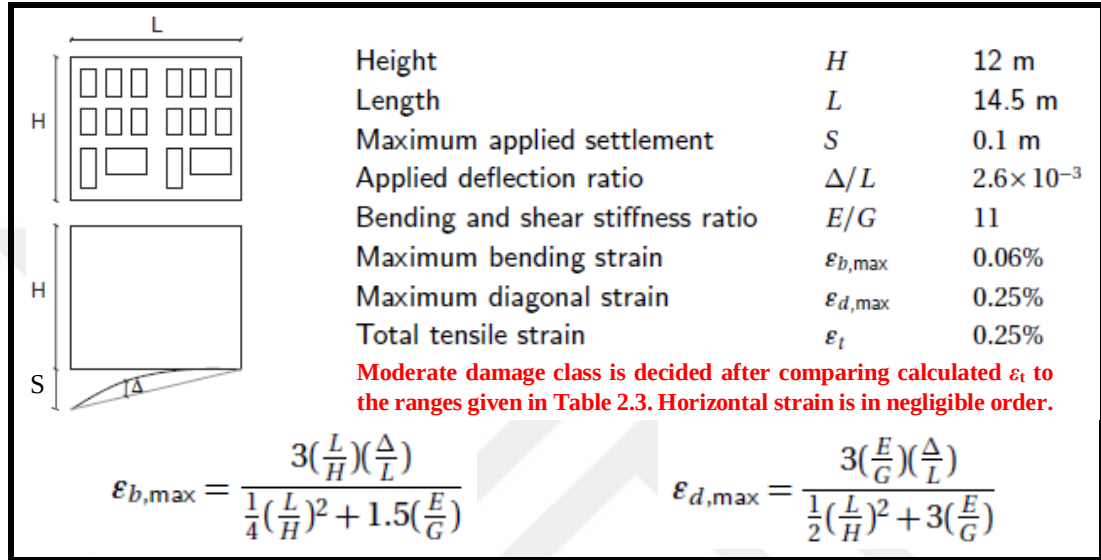


Figure A.1 : Analytical calculation of damage class for the masonry wall tested in Giardina et al. (2012).

APPENDIX B: Strain concentration around wall openings in Hoca Pasha Mosque case.

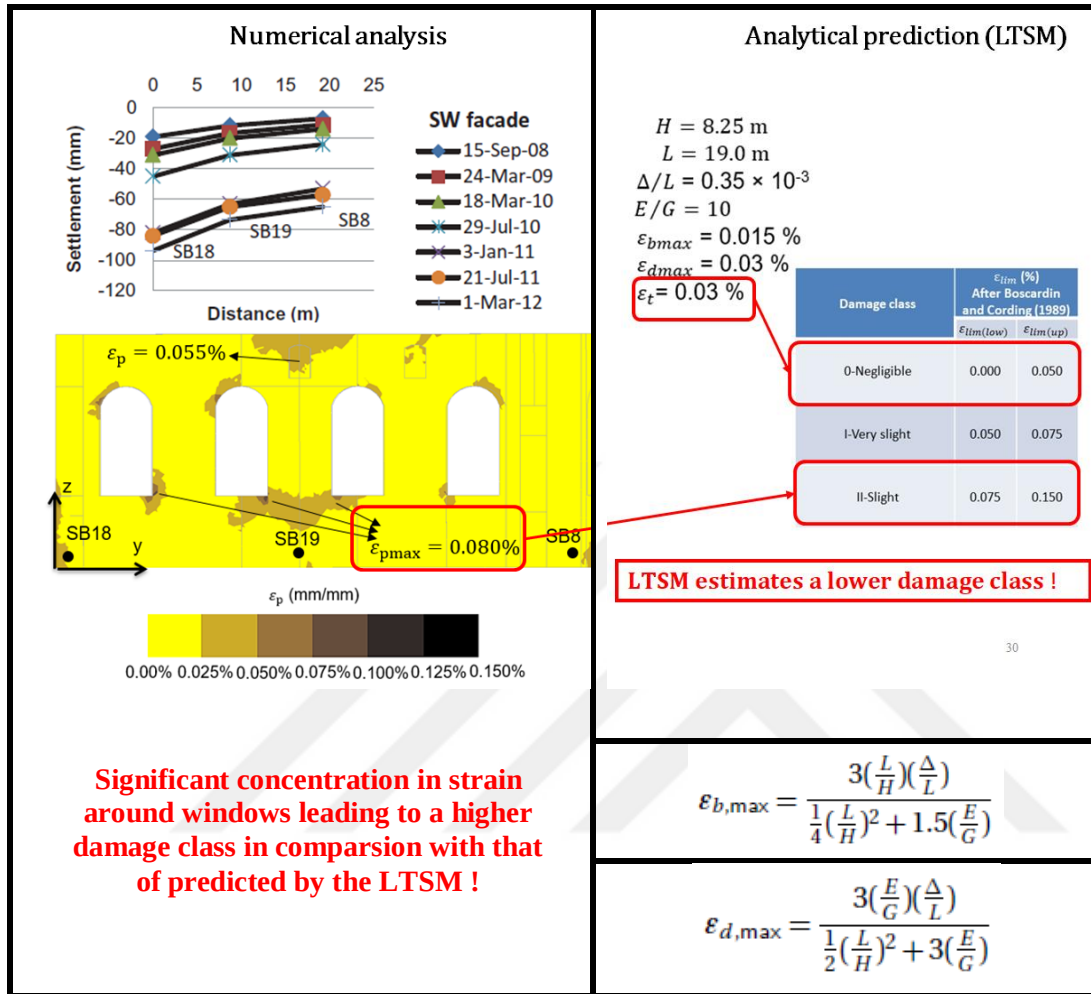


Figure B.1 : The comparison of the damage classes determined by the Finite Element Method and the LTSM for Hoca Pasha Mosque.

APPENDIX C: Comparisons between observed and numerically obtained crack and bedding interface relative displacements.

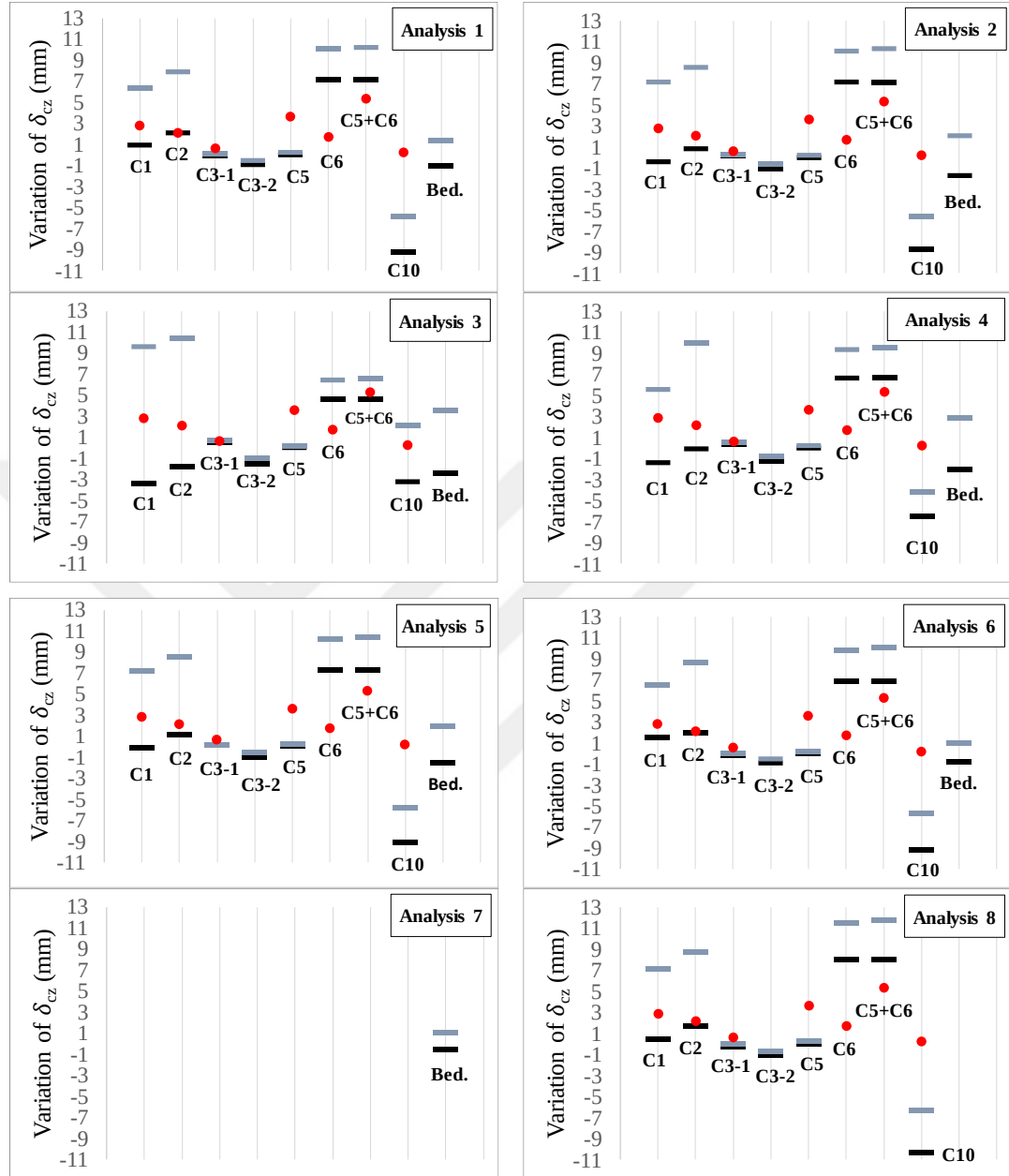


Figure C.1 : The ranges of variation of relative displacements of crack and bedding interface.



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PUBLICATIONS, PRESENTATIONS AND PATENTS ON THE THESIS:

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