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ΠΡΑΞΗ:

«ΘΑΛΗΣ- ΕΜΠ: ΠΡΩΤΟΤΥΠΟΣ ΣΧΕΔΙΑΣΜΟΣ ΒΑΘΡΩΝ ΓΕΦΥΡΩΝ ΣΕ ΡΕΥΣΤΟΠΟΙΗΣΙΜΟ ΕΔΑΦΟΣ ΜΕ ΧΡΗΣΗ ΦΥΣΙΚΗΣ ΣΕΙΣΜΙΚΗΣ ΜΟΝΩΣΗΣ»

MIS 380043

Επιστημονικός Υπεύθυνος: Καθ. Γ. ΜΠΟΥΚΟΒΑΛΑΣ

ΔΡΑΣΗ 7

Εφαρμογή σε Στατικώς Αόριστη Γέφυρα Ο.Σ.

ΠΑΡΑΔΟΤΕΑ:

Κριτήρια Επιτελεστικότητας

για Στατικώς Αόριστη Γέφυρα Ο.Σ. (Π7α)

Ιανουάριος 2015

Ευρωπαϊκή Ένωση
Ευρωπαϊκό Κοινωνικό Ταμείο
ΕΠΙΧΕΙΡΗΣΙΑΚΟ ΠΡΟΓΡΑΜΜΑ
ΕΚΠΑΙΔΕΥΣΗ ΚΑΙ ΔΙΑ ΒΙΟΥ ΜΑΘΗΣΗ
επένδυση στην κοινωνία της γνώσης
ΥΠΟΥΡΓΕΙΟ ΠΑΙΔΕΙΑΣ & ΘΡΗΣΚΕΥΜΑΤΩΝ, ΠΟΛΙΤΙΣΜΟΥ & ΑΘΛΗΤΙΣΜΟΥ
ΕΙΔΙΚΗ ΥΠΗΡΕΣΙΑ ΔΙΑΧΕΙΡΙΣΗΣ
ΕΣΠΑ 2007-2013
πρόγραμμα για την ανάπτυξη
ΕΥΡΩΠΑΪΚΟ ΚΟΙΝΩΝΙΚΟ ΤΑΜΕΙΟ
Με τη συγχρηματοδότηση της Ελλάδας και της Ευρωπαϊκής Ένωσης

Εκτενής Περίληψη

ΕΙΣΑΓΩΓΗ

Η παρούσα Τεχνική Έκθεση αποτελεί το **Παραδοτέο (Π7α)** του Ερευνητικού Προγράμματος με τίτλο:

ΘΑΛΗΣ-ΕΜΠ (MIS 380043)

Πρωτότυπος Σχεδιασμός Βάθρων Γεφυρών σε Ρευστοποιήσιμο Έδαφος με Φυσική Σεισμική Μόνωση

με Συντονιστή (Ερευνητικό Υπεύθυνο) τον Γεώργιο Μπουκοβάλα Καθηγητή ΕΜΠ.

Συγκεκριμένα, παρουσιάζονται τα αποτελέσματα της Δράσης Δ7 (που αφορούν στα κριτήρια επιτελεστικότητας), με τίτλο:

"Εφαρμογή σε Στατικώς Αόριστη Γέφυρα Ο.Σ.".

Το αντικείμενο του εν λόγω παραδοτέου περιγράφεται στην εγκεκριμένη ερευνητική πρόταση ως ακολούθως:

"Για ένα σύστημα στατικώς αόριστης γέφυρας οπλισμένου σκυροδέματος, οι επιτρεπόμενες κινήσεις της θεμελίωσης (καθιζήσεις και στροφές) θα καθοριστούν βάσει της αποδεχόμενης βλάβης και του επιπέδου λειτουργικότητας (π.χ. άνεση οδήγησης, επισκευάσιμη βλάβη, μη επισκευάσιμη βλάβη) καθώς και του αναμενόμενου επιπέδου σεισμικότητας (π.χ. σεισμική διέγερση περιόδου επαναφοράς 90, 450 ή 900 ετών). Οι επιτρεπόμενες κινήσεις της θεμελίωσης θα προκύψουν από την σύγχρονη αξιολόγηση των κατωτέρω.

- *Μία εκτεταμένη μελέτη των διαθέσιμων σχετικών κανονισμών και οδηγιών (π.χ. Ευρωκώδικας 2 – Μέρος 2, Ευρωκώδικας 8 – Μέρος 2, Ευρωκώδικας 7, MCEER & FHA – κεφάλαιο 11.4),*
- *περιπτώσεις απόκρισης πραγματικών γεφυρών κατά τη διάρκεια πρόσφατων σεισμών, και*
- *παραμετρικές (θεωρητικές και/ή) πειραματικές μελέτες διαφόρων τμημάτων γεφυρών (π.χ. βάθρα, εφέδρανα) υπό στατική και δυναμική φόρτιση."*

ΜΕΘΟΔΟΛΟΓΙΑ & ΑΠΟΤΕΛΕΣΜΑΤΑ

Η τυπική άνω διάβαση της Εγνατίας Οδού που υιοθετείται στο πλαίσιο της Δράσης 7 για τη μελέτη στατικώς αόριστης γέφυρας οπλισμένου σκυροδέματος, έχει συνολικό μήκος 99 m και αποτελείται από 3 ανοίγματα. Τα δύο ακραία ανοίγματα μήκους 27.0 m το καθένα και το μεσαίο μήκους 45.0 m εδράζονται επί δύο συμπαγών, κυκλικών βάθρων διαμέτρου 2.0 m, μονολιθικώς συνδεδεμένα με το κατάστρωμα. Η κατά μήκος του άξονα της γέφυρας κλίση 7% του προεντεταμένου καταστρώματος κιβωτιωειδούς διατομής διαμορφώνει τα ύψη των μεσοβάθρων σε 7.95 m και 9.35 m για το αριστερό και το δεξί βάθρο αντίστοιχα.

Η θεώρηση στατικώς αόριστου συστήματος και μη γραμμικής συμπεριφοράς επιβάλλει τη χρήση ανελαστικής στατικής ανάλυσης για τον ορισμό των επιτρεπόμενων μετακινήσεων της θεμελίωσης. Μέσω προσομοίωσης της γέφυρας με γραμμικά πεπερασμένα στοιχεία (SAP2000) και ορισμού των πιθανών πλαστικών αρθρώσεων (οδηγίες FEMA-356) στην κεφαλή και τον πόδα των δύο βάθρων, επιβάλλονται στην κατασκευή μόνιμα φορτία ($1.15\times G + 1.0\times P + 1.35\times Q$) και σταδιακά καθιζήσεις και στροφές στον πόδα των μεσοβάθρων μέχρι την κατάρρευση. Οι καθιζήσεις και οι στροφές που επιβάλλονται από τη ρευστοποίηση θεωρούνται τμήμα των μόνιμων δράσεων της κατασκευής μετά το σεισμό και λαμβάνουν τη μορφή $\Delta = \rho \pm \theta_{x,y}(\rho) \pm 0.3\theta_{x,y}(\rho)$ όπου οι στροφές συνδέονται άμεσα με την καθίζηση ($\theta_{x,y} = 0.05\cdot\rho$).

Τα αποτελέσματα της ανάλυσης σε συνδυαμό με τη βιβλιογραφική έρευνα ορίζουν τις τιμές των καθιζήσεων, συναρτήσει της πλαστιμότητας στροφής, οι οποίες οριοθετούν τις στάθμες βλάβης της κατασκευής. Για καθιζήσεις μικρότερες των 0.08 m η γέφυρα θεωρείται ότι συμπεριφέρεται ελαστικά και ακολούθως οι τιμές $0.08 \le \rho \le 0.15$ m ορίζουν τη στάθμη μικρής βλάβης, $0.15 \le \rho \le 0.20$ m τη στάθμη μεσαίας βλάβης ενώ τιμές άνω των 0.20 m οριοθετούν την κατάρρευση της κατασκευής. Επιλέγοντας για το σχεδιασμό μικρή αποδεκτή βλάβη η τιμή της επιτρεπόμενης καθίζησης του βάθρου ορίζεται σε 0.13 m λαμβάνοντας υπ᾽όψιν και ένα συντελεστή ασφάλειας 1.15.

ΟΜΑΔΑ ΕΡΓΑΣΙΑΣ

Η εργασία που παρουσιάζεται στο παρόν Παραδοτέο Π7α ολοκληρώθηκε με την συμβολή των παρακάτω μελών της **Ερευνητικής Ομάδας** του **Αριστοτέλειου Πανεπιστήμιου Θεσσαλονίκης (Τμήμα Πολιτικών Μηχανικών)**:

- **Ανδρέας Κάππος**, Καθηγητής
- **Αναστάσιος Σέξτος**, Αναπληρωτής Καθηγητής

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PROJECT:

«THALIS-NTUA: INNOVATIVE DESIGN OF BRIDGE PIERS ON LIQUEFIABLE SOILS WITH THE USE OF NATURAL SEISMIC ISOLATION»

MIS: 380043

Coordinator: PROF. G. BOUCKOVALAS

WORK PACKAGE 7

Application to Statically Indeterminate R.C. Bridges

DELIVERABLES

Performance Criteria

for Statically Indeterminate R.C. Bridges (D7a)

January 2015

European Union
European Social Fund
OPERATIONAL PROGRAMME
EDUCATION AND LIFELONG LEARNING
investing in knowledge society
MINISTRY OF EDUCATION & RELIGIOUS AFFAIRS, CULTURE & SPORTS
MANAGING AUTHORITY
NSRF 2007-2013
programme for development
EUROPEAN SOCIAL FUND
Co-financed by Greece and the European Union

1. Introduction

This Technical Report constitutes **Deliverable #7a** of the Research Project titled:

THALIS-NTUA (MIS 380043)

Innovative Design of Bridge Piers on Liquefiable Soils with the use of Natural Seismic Isolation

performed under the general coordination of Professor George Bouckovalas, NTUA (Principal Investigator). It presents the work and the corresponding results on tolerable ground deformations carried out for Work Package WP7, titled: "**Application to statically indeterminate RC bridges**"

The Scope of **Work Package WP7**, has been described in the approved Research Proposal as follows:

"The aim of this WP is to explore the feasibility of the proposed new design methodology, and the resulting advantages over conventional design methods, in the case of a statically indeterminate RC bridge (with continuous box-girder type deck). The main work tasks required to achieve this aim are the following:

(a) *Initially, the allowable foundation movements (settlements and rotations) will have to be established for a statically indeterminate RC bridge system, in terms of the afforded damage and serviceability level (e.g. driving discomfort, repairable damage, non-repairable damage) and the anticipated seismicity level (e.g. seismic excitation with 90, 450 or 900 years return period). The allowable foundation movements will result from a joint evaluation of:*

- *an extensive literature survey of relevant codes and guidelines (e.g. Eurocode 2-Part 2, Eurocode 8-Part 2, Eurocode 7, MCEER & FHA-chapter 11.4),*
- *examples of actual bridge performance during recent earthquakes, and*
- *parametric (theoretical and/or) experimental studies of various bridge components (e.g. piers, bearings) under static and cyclic-dynamic loading.*

(b) *Next, an actual river-bridge will be selected, with continuous deck system, large spans between piers (in excess of 40m) and extensive liquefiable soil layers underneath one or more of the bridge piers. Note that, following an initial survey, we have already identified a number of such bridges constructed as part of the Egnatia Highway in Northern Greece, such as the large bridge of Nestos River, with approximately 500m length, and a number of shorter bridges along the highway connection with the City of Serres. The piers of the selected bridge will be (re-) designed using the conventional foundation approach, i.e. pile groups with ground improvement between and around the piles.*

(c) *Finally, the static and seismic design of this bridge will be repeated with the new methodology of "natural" seismic isolation (i.e. shallow foundation and partial improvement, of the top part only of the liquefiable soil), in connection with the allowable foundation movements which were established in work task (1) above. The comparative advantages and limitations of the new design methodology, relative to the conventional one, will be consequently evaluated on the basis of technical as well as cost criteria.*

The work described herein corresponds to Work task (a) above. It has been performed with the contribution of the following members of the **Aristotle University of Thessaloniki (Department of Civil Engineering) Research Team**:

- **Andreas Kappos**, Professor
- **Anastasios Sextos**, Associate Professor

2. Literature Review

In the following, an overview is presented concerning existing allowable values of deformation of the foundation of bridges founded on spread footings.

2.1. Definitions

Barker et al. (1991) provide the definitions illustrated in **Figure 1,** concerning possible types of deformations (settlements) that may occur in bridges. According to their investigation, bridge deformations may appear in the form of *uniform settlement (ρ), uniform tilt (ω)* or *rotation (ϑ)* and *differential settlement (δ)*.

- ***Uniform settlement*** **(ρ)** is described as the rather theoretical situation where in each of the bridge foundations settles by the same amount. Even though no distortion of the superstructure occurs, excessive uniform settlement can lead to issues such as insufficient clearance at underpasses, as well as discontinuities at the juncture between approach slabs and the bridge deck, also referred to as "the bump at the end of the bridge" (Wahls, 1990) and inadequate drainage at the end of the bridge.
- ***Uniform tilt (ω)*** or ***rotation (ϑ)*** relates to settlements that vary linearly along the length of the bridge. Such type of deformation is most likely to occur in very stiff superstructures and single-span bridges. Usually, no distortion occurs in the superstructure, except in the case of non-monolithic connection between bridge components. In terms of traffic disturbance the same problems (bumps, drainage and clearance height) as mentioned above may occur.
- ***Non-uniform settlements,*** when the settlement at each support of a multi span bridge is different. It may be either regular or irregular, as noted in Figures 1(c) & 1(d). A regular pattern in deformation is characterized by a symmetrical distribution of settlement, from both ends of the bridge towards the center. In the irregular pattern, deformation is randomly distributed along the length of the bridge. The

non-uniform settlement of bridge foundations is also responsible for the onset of ***angular distortion (β)***, which affects the structural integrity of the superstructure. It is schematically described in Figures 1(c) and 1(d), and defined as:

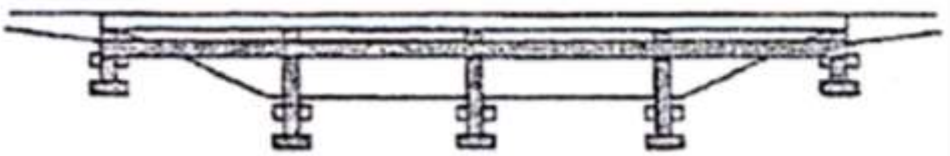
$$\beta = \frac{\delta}{S} \qquad [2.1]$$

where,

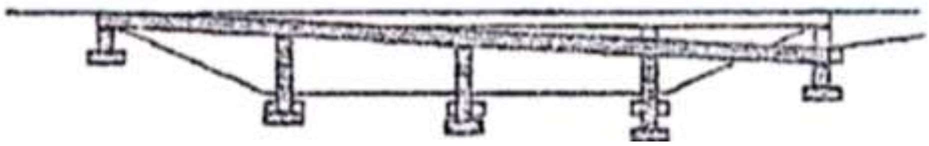
β = angular distortion (dimensionless)

δ = differential settlement between two consecutive foundations; in units of length

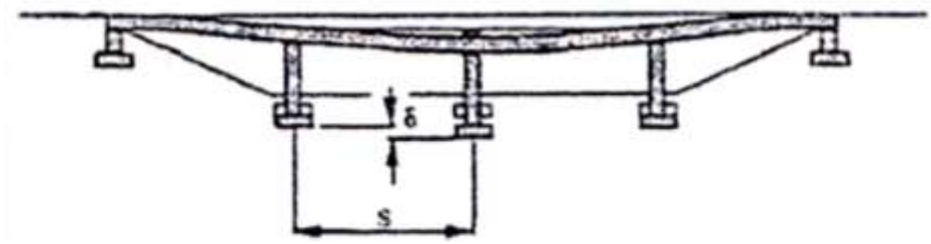
S = span length expressed in the same length units as the differential settlement.



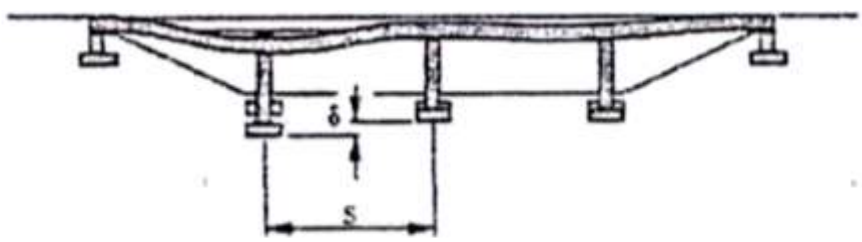
(a) Uniform Settlement (ρ).



(b) Uniform tilt (ω) or rotation (θ).



(c) Non uniform settlement (regular pattern of settlement).



(d) Non-uniform settlement (irregular pattern of settlement).

Figure2.1: Components of settlement and angular distortion in bridges (Barker et al., 1991).

Σχήμα 2.1. Ορισμός καθιζήσεων και γωνιακών παραμορφώσεων σε γέφυρες (Barker et al., 1991).

Differential settlements induce bending moments and shear in the bridge superstructure when the spans are continuous over supports. These moments and shears can potentially cause structural damage. ***Distress in the superstructure*** consists of cracks or other evidence of excessive stress in beams, girders, struts and diaphragms, as well as cracking and spalling of the deck. To a lesser extent, differential settlements can also cause damage to a bridge consisting of simple spans. However, the major concern with simple span bridges is the operational problems, i.e. inadequate drainage and insufficient clearance height at underpasses and mainly quality surface and aesthetics. Due to a lack of continuity over the supports, the changes in slope of the riding surface near the supports of a simple span bridge induced by differential settlements may be more severe than those in a continuous span bridge (Naresh et al, 2010).

In addition to the various types of settlements previously illustrated by Barker et al. (1991), ***horizontal displacement*** may also be induced in the foundation of bridges founded on spread footings. Excessive horizontal displacements may cause damage to the bearings and to the expansion joints of the bridge. Damage to bearings includes tilting or jamming of rocker bearings, as well as cases where rockers have pulled off the bearings, or where movement resulted in an improper fit between bearing shoes and rockers requiring repositioning. Elastomeric bearing pads are deformed, anchor bolts in the bearing shoes are sheared and cracking of concrete at the bearings is apparent. Other problems due to horizontally imposed displacement may involve horizontal movements occurring to the deck system, causing loss of the support of the deck or deck extending beyond the abutment and beams, jammed against the abutment, requiring to be cut. Sometimes, cutting of expansion joints may also be necessary (Moulton et al., 1985).

2.2. Movement Criteria

The selection of limiting values of imposed displacements constitutes a difficult issue to handle, due to a great number of factors affecting them, namely the type of structure (type of spans, length and stiffness of spans), the type of construction material, the type of ground, the proposed use of the structure, the confidence with which the acceptable value of the movement can be specified, the occurrence and rate of ground movements, and so on.

On the other hand, the limit between tolerable and non-tolerable movement is often difficult to discern, and may depend on factors other than the physical condition of the bridge, such as the cost and practical problems involved in repair and maintenance. Generally, the definition of non-tolerable damage proposed by the Transportation Research Board's Committee A2K03 on "*Foundations of bridges and other structures*" is adopted: *"Movement is **not** tolerable if damage requires costly maintenance and/or repairs **and** a more expensive construction to avoid this would have been preferable".*

2.3. Literature survey

In the following, a literature review is made of the existing allowable values of deformation under **static** loading. The results are mainly based on field studies of numerous existing bridges founded on spread footings. This outline provides useful insight as to the order of magnitude and the type of such deformations as well as, to their effect on the serviceability and the structural integrity of bridges.

Bozozuk (1978) attempted to distinguish tolerable from non-tolerable displacements for abutments and piers founded on spread footings. His survey involved 120 cases of spread footings, treating all types and sizes together. He classified displacements as tolerable, when the maintenance needs of the bridge are moderate, despite the magnitude of the displacements, and as non- tolerable when considerable maintenance and repair works are required. The work of Bozozuk (1978) was published at the same time as those of Walkinshaw (1978) and Grover (1978) and was documented via an extensive research on allowable displacements undertaken in the USA and Canada and published by the Transportation Research Board (TRB). Therefore Bozozuk's definition of tolerable and non-tolerable displacements also applies to the limiting values proposed by Walkinshaw and Grover (see Table 1).

DiMillio (1982) attempted to evaluate the behaviour of 148 highway bridges supported by spread footings on engineered fills, in conjunction with detailed survey investigations of the foundation movement of 28 selected bridges. It was found that bridges easily tolerated differential settlements of 1 to 3 inches (25 to 75 mm) without significant distress, especially when high embankments of good quality borrow materials are constructed over firm foundation soils.

Table 2.1: Engineering performance of bridges on spread footings.

Πίνακας 2.1: Κριτήρια επιτελεστικότητας γεφυρών με επιφανειακές θεμελιώσεις.

Type of deformation	Magnitude of deformation	Damage Level	Reference
Settlement ρ_V (mm)	< 50	Tolerable	Bozozuk (1978)
	63	Harmful but tolerable (Ride quality)	Walkinshaw (1978)
	25.4 - 76.2	Harmful but tolerable	DiMillio (1982)
	50 - 100	Harmful but tolerable	Bozozuk (1978)
	> 63	Structural damage	Walkinshaw (1978)
	> 100	Intolerable	Bozozuk (1978)
	102	Intolerable (Ride quality and structural damage)	Grover (1978)
	>102	Intolerable (for abutments)	Wahls (1990)
Horizontal displacement ρ_H (mm)	< 25	Tolerable	Bozozuk (1978)
	25.4 - 50.8	Harmful but tolerable	Moulton et al. (1985)
	25 - 50	Harmful but tolerable	Bozozuk (1978)
	50	Structural damage	Walkinshaw (1978)
	> 50	Not tolerable (Ride quality and structural damage)	Bozozuk (1978)
	> 51	Intolerable (for abutments)	Wahls (1990)

Moulton et al. (1985) carried out a survey that was based on a nationwide examination of 314 concrete and steel bridges on spread footings in the USA and Canada. In this study, an effort was made to provide information regarding the possible structural damage induced by excessive vertical and horizontal displacement. The definition for non-tolerable damage proposed by the TRB Committee A2K03 was adopted. The results were classified according to the type of spans, the length and stiffness of spans, and the type of construction material. It was shown that many highway bridges can tolerate significant magnitudes of total and differential vertical settlement without being seriously overstressed, sustaining serious structural damage, or suffering impaired riding quality. In particular, it was found that a longitudinal angular distortion (equation 1) of 0.004 would most likely be tolerable for continuous bridges of both steel and concrete, while a value of angular distortion of 0.005 would be a more suitable limit for simply supported bridges. In this project, it was also pointed out that in the case of coexistence of vertical and horizontal movements, the tolerable horizontal movement should be limited to 25mm, while in the case where the vertical displacement is small, the tolerable horizontal movement can be increased by 50%.

Table 2.2: Proposed Serviceability Criteria for bridges by various researchers.

Πίνακας 2.2: Κριτήρια λειτουργικότητας γεφυρών όπως προτείνονται από διάφορες επιστημονικές ομάδες.

Type of deformation	Magnitude of deformation	Bridge type	Reference
Angular Distortion β	0.004 (1/250)	Continuous steel/concrete bridges with l≥15.24m(50ft)	Moulton et al. (1985)
	0.005 (1/200)	Simply-supported steel/concrete bridges withl≥15.24m(50ft)	Moulton et al. (1985)
	1/250	Continuous bridges (Bridge abutment)	Wahls (1990) Stark et al. (1995)
	1/200	Simply-supported bridges (Bridge abutment)	Wahls (1990) Stark et al. (1995)
Differential Settlement Δρ (mm)	< 76.2	Bridge abutment for bridge lifetime (steel & concrete bridges)	Moulton et al. (1985)
	< 50.8	Bridge pier for bridge lifetime (steel &concrete bridges)	Moulton et al. (1985)
	< 50.8	Bridge abutment following bridge completion (steel &c oncrete bridges)	Moulton etal. (1985)
	< 31.75	Bridge pier following bridge completion (steel bridges)	Moulton et al. (1985)
	< 38.1	Bridge pier following	Moulton et

		bridge completion (concrete bridges)	al. (1985)
Horizontal displacements (mm)	<38	Acceptable	Moulton et al. (1985)
Horizontal along with vertical displacements (mm)	<25	Acceptable	Moulton et al. (1985)

According to their surveys, Wahls (1983) and (1990) and Stark et al. (1995) arrived to the conclusion that angular distortions of 1/250 of the span length for continuous spans and 1/200 for simply-supported spans were considered acceptable. Moreover, differential movements not greater than 50 mm laterally and less than 100 mm vertically, appear to be tolerable, assuming that approach slabs or other provisions are made to minimize the effects of any differential movements between abutments and approach embankments.

Engineering performance of bridges examined in the aforementioned studies, in terms of vertical and horizontal displacements of abutments and piers are illustrated in Table 2.1, listed in ascending magnitude of deformation. In Table 2.2, proposed serviceability criteria for bridges by the aforementioned researchers are summarized.

2.4. Provisions of Codes

Codes, currently in effect in Europe and other areas (Eurocodes, AASHTO, etc.), do not directly correlate the desired performance of a bridge (performance levels) to limiting values of measurable deformations either of the structure or the foundation. However, the approach adopted by the Codes is that the desired behaviour of a structure (in terms of service and damage level) becomes more demanding, as the importance of the structure and the probability of an earthquake increase. The requirement for a specific behaviour of a bridge under static and dynamic actions is today indirectly satisfied, when the structure is designed for two limit states, the Ultimate Limit State (ULS) and the Serviceability Limit State (SLS).

Ultimate Limit State (ULS) is associated with the safety of people and/or the loss of the bearing capacity of the structure. This limit condition can occur either due to structural failure or a failure of the soil.

Serviceability Limit State (SLS) is related to the functionality and service requirements of a structure to ensure adequate performance under expected conditions. Conditions of total collapse are not involved here. Nevertheless, conditions are examined, which prevent the intended use of the structure and criteria are set concerning deformations affecting the appearance and the comfort of the users, vibrations that cause discomfort to people or restrict the operationality of the structure and finally damage that affects the appearance, durability or the function of the project.

According to AASHTO (2002) and AASHTO – LRFD (2007 and 2009 Interims), for bridges on spread footings, movement of foundations in both vertical and lateral directions shall be investigated in the frame of Serviceability Limit State, i.e. settlements and/or horizontal displacements, as well as the angular distortion caused by differential settlements of adjacent footings. Design shall be based on rideability and cost criteria. Immediate settlement shall be determined using the service load combinations while for time-dependent settlements only the permanent loads shall be taken into account. Concerning proposed limiting values for movement of footings, appropriate criteria should be developed, consistent with the function and type of structure, anticipated service life and consequences of unacceptable movements on structure performance and should be established by empirical procedures or structural analysis. Due to the complexity of the phenomenon, only in the comments of the Code, limiting values are suggested for angular distortion (δ/s) between adjacent foundations, as a function of the structural system of the bridge, namely 0.008 for simple span bridges and 0.004 for continuous span bridges. For rigid frames special analyses are required (see Table 2.3). These limits lead to relatively high values of acceptable differential settlements, for example for a span of 30 m a differential settlement of 120 mm for a continuous span and 240mm of a simple span are acceptable. It should be noted here that such high values of differential settlements create concern for structural designers who often arbitrarily limit the criteria to one-half to one-quarter of the suggested values, not so much for reasons related to the structural integrity of the bridge but mainly for practical reasons based on the tolerable limits of deformation of another structures associated with a bridge e.g. approach slabs , wing walls, pavement structures, drainage grades, utilities of the bridge, deformations that adversely affect quality of ride, etc. (Naresh et al., 2010). That is why, the suggested criteria should be considered in conjunction with functional or performance criteria not only for the bridge structure itself but for all the associated facilities as well.

Finally, according to AASHTO, when designing against seismic actions in the frame of the ultimate limit state, foundation movements are not taken into account. In Division I-A of the Code, referring to the design of foundations in seismically active areas, it is pointed out that special consideration should be given to the potential settlement of footings on sand, resulting from ground motions induced by earthquake loadings.

A similar approach is also followed in Eurocodes. According to Eurocode 2 for Bridges (EN1992-2:20050, the effects of uneven settlements of the structure due to soil subsidence should be considered for the verification of serviceability limit states. Concerning ultimate limit states, they should be considered only where they are significant, for example where second order effects are of importance. In other cases for ultimate limit states they need not be considered, provided that the ductility and rotation capacity of the elements are sufficient.

Moreover, according to EC1997-1:2004, the assessment of the behaviour of bridges on shallow foundations involves both displacement of the entire foundation and differential displacements of parts of the foundation. Specifically, as suggested in Appendix H of the Code, the following components of foundation movement should be considered: settlement, relative (or differential) settlement, rotation, tilt, relative deflection, relative rotation, horizontal displacement and vibration amplitude. According to the code, any differential movements of foundations leading to deformation in the supported structure should be limited to ensure that they do not lead to a limit state in the supported structure, and this is achieved when design values remain lower of certain limiting values. As limiting value for a particular deformation is defined the value of the deformation at which a serviceability limit state, such as unacceptable cracking etc., is deemed to occur in the supported structure (§ 2.4.8). As noted in the Code, selection of design values for limiting movements and deformations is not an easy task and should take into account various factors, such as the type of structure, the type of construction material, the type of foundation, the services entering the structure, etc. Thus, certain limiting values are not provided and it is suggested that they should be agreed during the design of the supported structure.

However, in the absence of specified limiting values of structural deformations of the supported structure, it is proposed that for normal, routine, structures the values of structural deformation and foundation movement given in Annex H may be used. More specifically, to prevent the occurrence of a serviceability limit state in the structure, permissible values of relative rotations of various types of structures could range from1/2000 to about 1/300, while a maximum relative rotation of 1/500 is judged as acceptable for many structures. The relative rotation likely to cause an ultimate limit state is proposed to be 1/150. For normal structures with isolated foundations, total settlements up to 50 mm are often acceptable. Larger settlements may be acceptable provided the relative rotations remain within acceptable limits and provided the total settlements do not cause problems with the services entering the structure, or cause tilting etc. (Table 2.3). On the other hand, according to section 6.5.5, an ultimate limit state due to differential vertical and horizontal foundation displacements could be avoided by adopting appropriate prescriptive measures.

Table 2.3: Tolerable movement criteria for bridges proposed by various Codes.
Πίνακας 2.3: Κριτήρια ανεκτών μετατοπίσεων γεφυρών όπως προτείνονται από διάφορους κανονισμούς.

Code	Type of deformation	Magnitude of deformation	Bridge type	Limit State
AASHTO 2002, 2007 with 2009 Interims	**Angular Distortion β**	0.004 (1/250)	Continuous	Serviceability
		0.008 (1/125)	Simply-supported	
EN1997-1 (Annex H)	**Angular Distortion β**	0.002 (1/500)	all normal, routine structures	Serviceability
		1/150	all normal, routine structures	Ultimate
	Total settlement	50 mm	normal structures with isolated foundations	Serviceability

According to EN 1998-2:2005, the desired behaviour of a bridge against seismic actions is qualitatively defined in terms of service and damage level after the seismic event, as a function of the importance of the bridge and the probability of the earthquake. For the Ultimate Limit State, the bridge is implicitly anticipated to preserve its structural integrity and hold adequate residual resistance in order to avoid total collapse. Considerable damage is expected to occur, mainly in the form of flexural yielding of specific sections (i.e. the formation of plastic hinges) in the piers, which in the absence of seismic isolation is a desirable situation. The bridge deck should in general be designed to avoid damage, except for breakage of secondary components, such as expansion joints and continuity slabs. Also, the bridge deck must be able to accommodate loads from piers experiencing plastic hinging and must not become unseated under extreme seismic displacement. In the case of rare seismic actions, the parts of the bridge contributing to energy dissipation should be designed to enable emergency traffic and inspections in the post-earthquake period and to be easily repairable. For the Serviceability Limit State, a high probability of occurrence seismic scenario may cause only minor damage to secondary components and to contributing to energy dissipation parts of the bridge. All other components of the bridge are expected to remain intact; traffic should not be disturbed and repairs should not be urgent. Although the design seismic criteria proposed in the Code aim explicitly at satisfying the no-collapse requirement, they implicitly cover the damage minimization requirement as well.

Further, as it is noted in EN1998-2, § 5.5, the aforementioned requirements are satisfied for ULS (and consequently for SLS as well), by verifying the structure against seismic combinations that do not include action effects due to imposed deformations caused by settlements of supports or residual ground movements due to seismic faulting. An exception to this rule is the case of bridges in which the seismic action is resisted by elastomeric laminated bearings, where elastic behaviour of the system shall be assumed and the action effects due to imposed deformations shall be accounted for. In the code, no limiting values for foundation movements under seismic conditions are proposed.

2.5. Other approaches

As an alternative to the previous, other approaches may be adopted to specify limiting values for foundation movements of bridges.

According to the Japanese method JBDPA ’90-91 which is a method for the post-earthquake inspection and rapid damage assessment of buildings, a damage classification is attempted according to the maximum inclination of the building after a certain event (Rossetto et al., 2010). The classification according to the inclination angle θ is illustrated in Table 2.4. Although the method refers to the damage assessment of buildings, the magnitude of the inclination angle of the piers may also be considered as a criterion for the damage assessment of bridges. To this end, a limiting value of 0.02 rad may be accepted for the serviceability limit state.

Finally, according to FEMA-356, four discrete Structural Performance Levels related to certain post-earthquake damage states, are defined for buildings:

- Immediate Occupancy (S-1), defined as the post-earthquake damage state that remains safe to occupy, essentially retains the pre-earthquake design strength and stiffness of the structure,
- Life Safety (S-3), defined as the post-earthquake damage state that includes damage to structural components but retains a margin against onset of partial or total collapse,
- Collapse Prevention (S-5), defined as the post-earthquake damage state that includes damage to structural components such that the structure continues to support gravity loads but retains no margin against collapse, and
- Not Considered (S-6), defined as the post-earthquake damage state where a building rehabilitation does not address the performance of the structure.

Table 2.4: Damage classification according to JBDPA 90-91 (Rossetto et al., 2010).
Πίνακας 2.4: Κατηγοριοποίηση βλαβών σύμφωνα με την Ιαπωνική μέθοδο JBDPA 90-91 (Rossetto et al., 2010).

Type of deformation	Magnitude of deformation	Damage level
Inclination angle θ (rad)	<0.01 0.01 -0.03 0.03- 0.06 > 0.06	Small Moderate Severe Collapse

Appropriate acceptance criteria relate these Structural Performance Levels to limiting damage states for vertical elements of lateral-force-resisting systems, in terms of drift values. The drift values proposed by FEMA are presented in Table 2.5 and are discerned into transient and permanent. They are typical values provided to illustrate the overall structural response associated with various Structural Performance Levels. In this sense, these values may also be adopted as limiting drift values for piers of bridges.

Table 2.5: Structural Performance Levels and damage for common vertical elements of lateral-force-resisting systems of buildings according to FEMA-356, Table C1-3.

Πίνακας 2.5: Επίπεδα επιτελεστικότητας για τυπικά κατακόρυφα στοιχεία κτιρίων σύμφωνα με τις οδηγίες της FEMA-356, Πίνακας C1-3.

Type of deformation	Magnitude of deformation	Structural Performance Level
Drift **φ (rad)** **Concrete Frames**	4% transient or permanent 2% transient; 1% permanent 1% transient; negligible permanent	**Collapse Prevention** **S-5 Life Safety** **S-3 Immediate Occupancy**
Drift **φ (rad)** **Concrete Walls**	2% transient or permanent 1% transient; 0.5% permanent 0.5% transient; negligible permanent	**Collapse Prevention** **S-5 Life Safety** **S-3 Immediate Occupancy**

3. Application to a statically indeterminate RC bridge

For the case of statically indeterminate reinforced concrete (RC) bridge studied in Work Package 7 of this research project, a typical three-span overpass of Egnatia Highway with a total length of 99 m is adopted. The two outer spans of length 27.0 m each and the middle span of 45.0 m are supported by the two solid circular 2.0 m diameter piers, monolithically connected to the deck. The 7% slope of the prestressed concrete box girder along the bridge axis results in 7.95 m and 9.35 m height for the left and the right pier, respectively.

3.1. State of the art in bridge performance

Limiting values of imposed displacements in bridge performance are generally not directly associated with specific limit states of the structure due to a great number of factors affecting them (type of structure, construction material, soil, use of the structure etc.). Only simplified approaches from the literature are available wherein allowable values of deformation are provided mainly for static loading.

According to early researchers (Table 2.1), vertical displacements - settlements up to 5.0 cm are considered *tolerable*, a value associated with the Serviceability Limit State, while settlements between 5.0 cm and 10.0 cm are considered *harmful but tolerable,* a condition befitting to Ultimate Limit State. Settlements greater than 10.0 cm are considered *intolerable*, a rather conservative value for new structures. A smaller value, about 40 mm, is proposed for the differential settlement of concrete bridge piers by Moulton et al. (1985) as a serviceability criterion (Table 2.2).

Under the assumption that the settlement of the abutment is practically zero, limiting values of differential settlements divided with the span length result to allowable values of angular distortion. Based on Moulton et al. (1985), the allowable angular distortion for continuous concrete bridges equals to 1/250 (Table 2.2), corresponding, for the case studied, to differential settlement of 10.8 cm considering the 27.0 m span of the bridge. A similar approach is also adopted by codes, where AASHTO is setting the same limit of 1/250 for the allowable angular distortion of continuous bridges (Table 2.3) in the serviceability limit state. Finally, EN1997-1 sets for all normal, routine structures a serviceability limit state value of 1/500 that corresponds, in the case examined, to 54 mm of differential settlement and 1/150 for ultimate limit state, corresponding to 180 mm of differential settlement.

In conclusion, it is difficult and even risky to provide ‘all-purpose’ limits for bridges. Thus, the maximum tolerable displacements and rotations should be calculated for each individual structure through sensitivity analyses, as made herein. Due to the statically indeterminate system of the bridge and the assumption of the non-linear behaviour, the above limits are defined through inelastic static analysis by setting increasing settlements and rotations to the base of the piers.

3.2. Tolerable settlements and rotations for the statically indeterminate bridge system

The methodology proposed by Bouckovalas et al. (2014a) addresses the design of spread foundations on a surface “crust” of non-liquefiable, natural or improved ground with sufficient thickness, shear strength and lateral extent, so that the seismic response of the raft is satisfactory. The liquefiable susceptible soil layer is allowed to liquefy reducing the inertia forces acting on the superstructure, hence providing a ‘natural’ seismic isolation.

The shallow foundation has to be designed at every pier based on the support-dependent soil properties and the pertinent design parameters. The latter are related to the soil properties, soil geometry of the liquefiable and the improved zone, earthquake ground motion properties, predominant period and number of cycles, footing geometry, as well as the maximum tolerable displacements and rotations. The latter will be defined at this stage ensuring the Immediate Use/Occupancy performance level and, for stronger ground motions with larger deformations, the acceptable (repairable) damage; according to Eurocode, the concept of “Life Protection” or else the “Limit State of Significant Damage”.

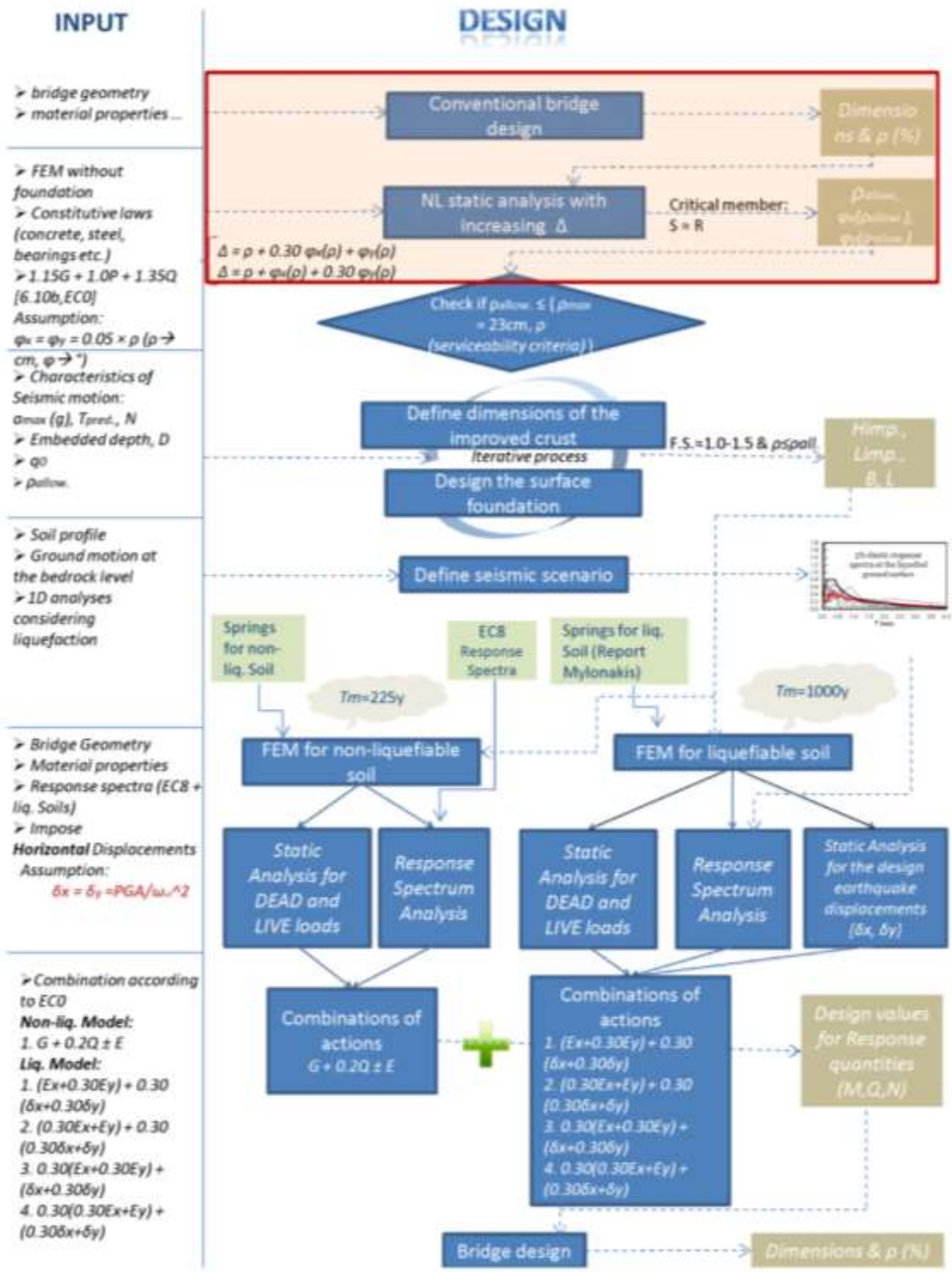


Figure 3.1. Flow chart of the proposed methodology for bridge design on liquefaction susceptible soils.

Σχήμα 3.1.Διάγραμμα ροής της προτεινόμενης μεθοδολογίας σχεδιασμού γεφυρών σε ρευστοποιήσιμα εδάφη.

The finite element program SAP2000 was used for the inelastic static analysis of the overpass with a view to defining the allowable settlement. Elastic beam elements were used for both the deck and the piers. End regions of both piers (head and base) were considered as the potential plastic hinge zones, hence capable of undergoing plastic rotations when yield moment (M_y) is exceeded. Hinges were then assigned to both ends of the frame elements. The hinge properties were calculated based on FEMA-356 guidelines.

Gravity (G) and live (Q) loads were applied to the bridge according to the combination of actions defined in Equation 6.10b of the EC0 (EN1990:2005):

Combination of actions:

$$\xi \times \gamma_G \times G + \gamma_P \times P + \gamma_Q \times Q = \quad [3.1]$$
$$= 0.85 \times 1.35 \times G + 1.0 \times P + 1.35 \times Q =$$
$$= 1.15 \times G + 1.0 \times P + 1.35 \times Q$$

Nonlinear static analysis was then performed applying a predefined pattern (Δ) of displacements (settlements, ρ) and rotations (around the x (ϑ_x) and y (ϑ_y) bridge axes) at the base of the piers until the bridge collapses. This pattern was defined assuming that settlements and rotations triggered from the liquefaction phenomenon will act as permanent loads in the structure after the earthquake. The following combinations were applied:

- $\Delta = \rho \pm \theta_y(\rho) \pm 0.3\theta_x(\rho)$ [3.2]
- $\Delta = \rho \pm \vartheta_x(\rho) \pm 0.3\vartheta_y(\rho)$

Rotations ϑ_y and ϑ_x were defined as a function of the imposed settlement ρ according to the empirical equation:

$$\vartheta_x = \vartheta_y = 0.05 \cdot \rho \quad [3.3]$$

where rotations ϑ_y and ϑ_x are expressed in deg while settlement ρ is expressed in cm. Figure 3.2 shows all the examined load patterns (32) applied at the base of the two piers for the estimation of the tolerable settlement.

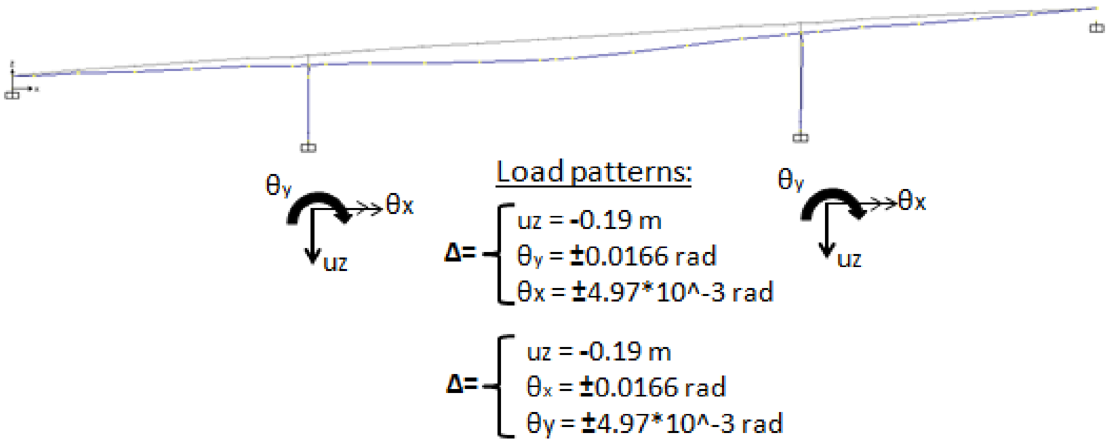


Figure 3.2. Examined load patterns applied at the base of the piers for the estimation of the tolerable settlement.

Σχήμα 3.2. Εξεταζόμενες φορτιστικές καταστάσεις επιβαλλόμενες στους πόδες των βάθρων για τον υπολογισμό της επιτρεπόμενης καθίζησης.

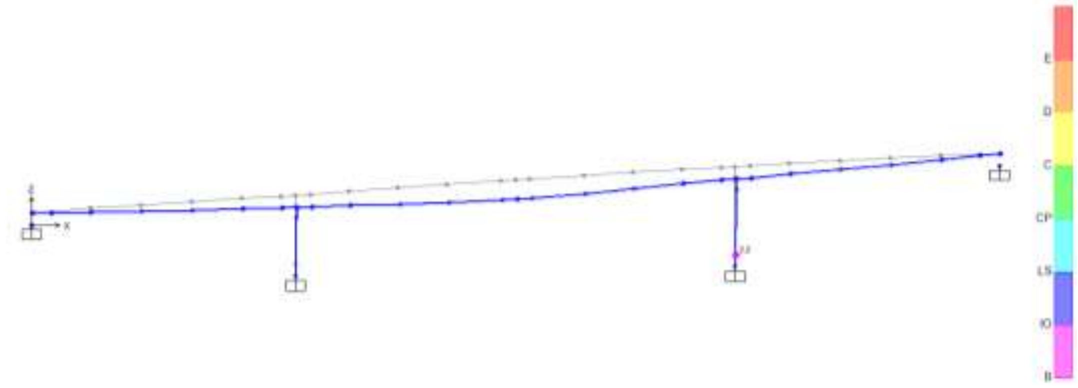
The most critical combination of settlements and rotations is summarized in Table 3.1. This combination corresponds to antisymmetric rotations ϑ_y at the base of the two piers.

Table 3.1: Critical combination of the applied settlements and rotations at the base of the two piers.

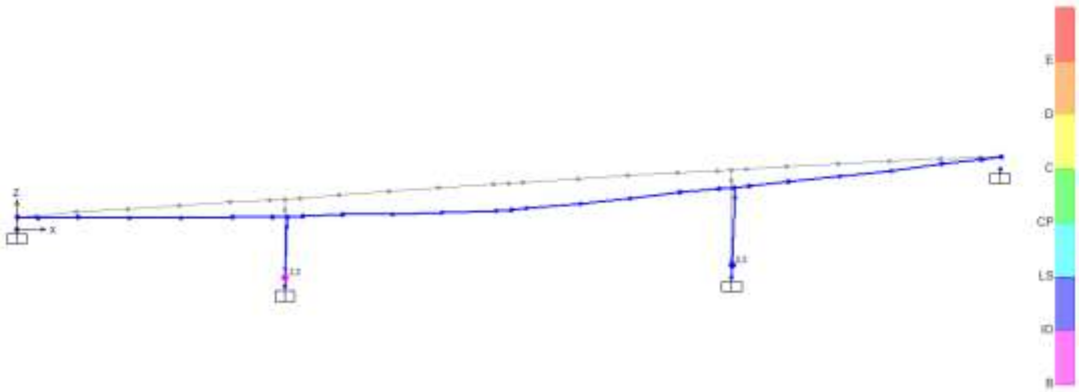
Πίνακας 3.1: Κρίσιμος συνδυασμός των επιβαλλόμενων καθιζήσεων και στροφών στις βάσεις των δύο βάθρων.

Bridge Pier	**Applied Δ**
Left	$\rho + \vartheta_y(\rho) + 0.3\vartheta_x(\rho)$
Right	$\rho - \vartheta_y(\rho) - 0.3\vartheta_x(\rho)$

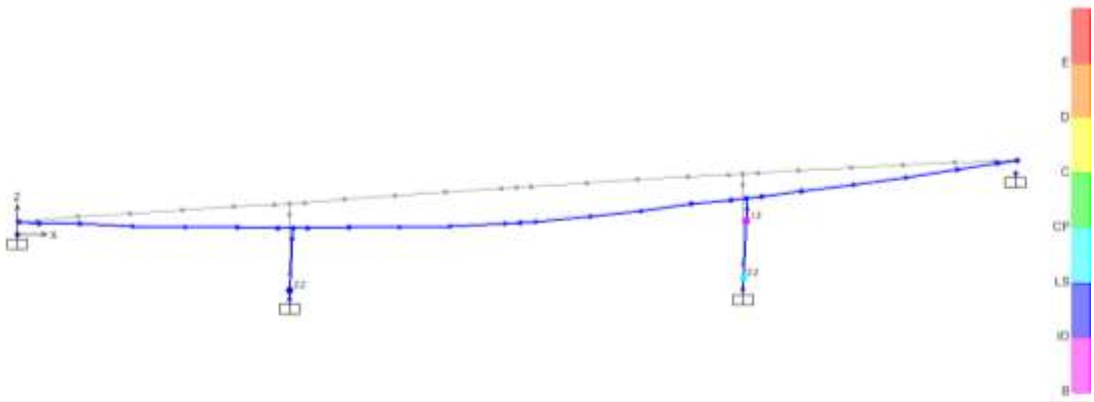
By gradually increasing the imposed combination of loads (Table 3.1), it was observed that the first plastic hinge is formed at the base of the right pier corresponding to a settlement of 0.10m. A plastic hinge is then formed at the base of the left pier and finally the bridge collapses when a plastic hinge is formed at the top of the right pier. This corresponds to a settlement of 0.21 m. Figure 3.3 presents the sequence of plastic hinge formation in the examined overpass.



(a) First plastic hinge at right pier base.



(b) Second plastic hinge at left pier base.



(c) Third plastic hinge at right pier top.

Figure 3.3. Plastic hinge formation in the structure.

Σχήμα 3.3. Πορεία σχηματισμού πλαστικών αρθρώσεων στη γέφυρα.

The inelastic static analysis results are presented in terms of settlement (ρ) as a function of the rotational ductility (μ_ϑ). To compute μ_θ, the chord rotation at yielding as well as the plastic rotation is needed. The chord rotation at yielding is derived according to Eq. A.10b of the EC8 (EN1998-3:2005):

$$\theta_y = \varphi_y \cdot \frac{L_v + \alpha_v \cdot z}{3} + 0.0013 \cdot \left(1 + 1.5 \cdot \frac{h}{L_v}\right) + 0.13 \cdot \varphi_y \cdot \frac{d_b \cdot f_y}{\sqrt{f_c}} \quad [3.4]$$

where:

φ_y	is the yield curvature of the end section,
$L_V = M/V$	is the ratio moment/shear at the end section,
f_y and f_c	are the steel yield stress and the concrete strength, respectively both in MPa,
d_b	is the (mean) diameter of the tension reinforcement.
$a_v = 0$	
z	is the length of internal lever arm, taken equal to d-d' in columns
d and d'	are the depths to the tension and compression reinforcement, respectively.

Yielding curvature φ_y is defined from section analysis performed in Opensees. Figure 3.4 shows the moment-curvature relationship (actual and bilinear) derived through section analysis for the base of the two conventionally designed piers. The shear ratio M/V is extracted from SAP2000.

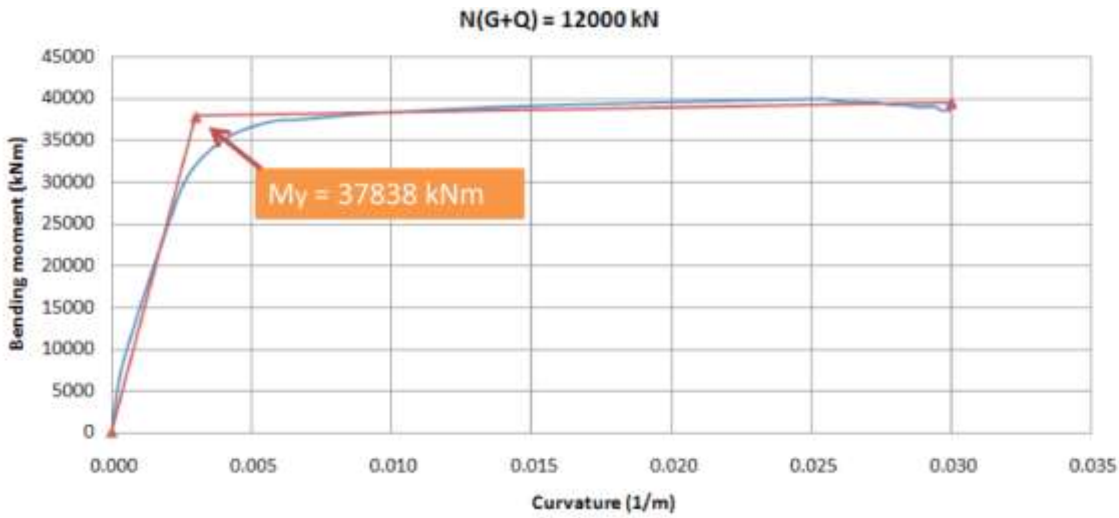


Figure 3.4. Moment-curvature relationship (actual and bilinear) derived through section analysis for the the base of the two conventionally designed piers.

Σχήμα 3.4. Σχέση καμπτικής ροπής – καμπυλότητας (πραγματικής και διγραμμικά εξιδανικευμένης) η οποία προέκυψε από ανάλυση διατομής για τον πόδα των δύο βάθρων που σχεδιάστηκαν ακολουθώντας τη συμβατική μεθοδολογία σεισμικού σχεδιασμού γεφυρών.

The chord rotation at yielding ϑ_y of the right pier is then calculated by substituting in Equation 3.4:

$$\theta_y = 0.003\frac{38841/8539}{3} + 0.0013\cdot\left(1 + 1.5\cdot\frac{2.0}{38841/8539}\right) + 0.13\cdot 0.003 \cdot\frac{0.025\cdot 500}{\sqrt{27.5}} = 0.00769\,rad$$

Plastic rotation ϑ_p was automatically computed by SAP2000. Figure 3.5 presents the applied settlements at the base of piers as a function of the rotation ductility.

At this stage, in order to define the tolerable settlements, performance criteria should be established. Generally speaking, it is up to the designer to define these criteria associated with the acceptable damage at the bridge. The performance criteria adopted in this case (in the light of the state-of-the-art presented in section 2) are presented in Figure 3.5. Specifically, for settlements (ρ) smaller than 0.08m, no damage is expected in the bridge as it responds in the elastic range. For settlements in the range 0.08 ≤ ρ ≤ 0.15m, minor damage is expected, while for settlements in the range 0.15 ≤ ρ ≤ 0.20m, moderate damage is expected. Finally, the bridge 'collapses' for imposed settlements greater than 0.20m. It was decided to design the bridge based on the performance criterion corresponding to minor damage, hence a value of 0.15m can be tolerable for the settlements at the base of piers. By further assuming a safety factor of 1.15 (according to Equation 3.1), the tolerable settlement is set to 0.15/1.15=0.13m.

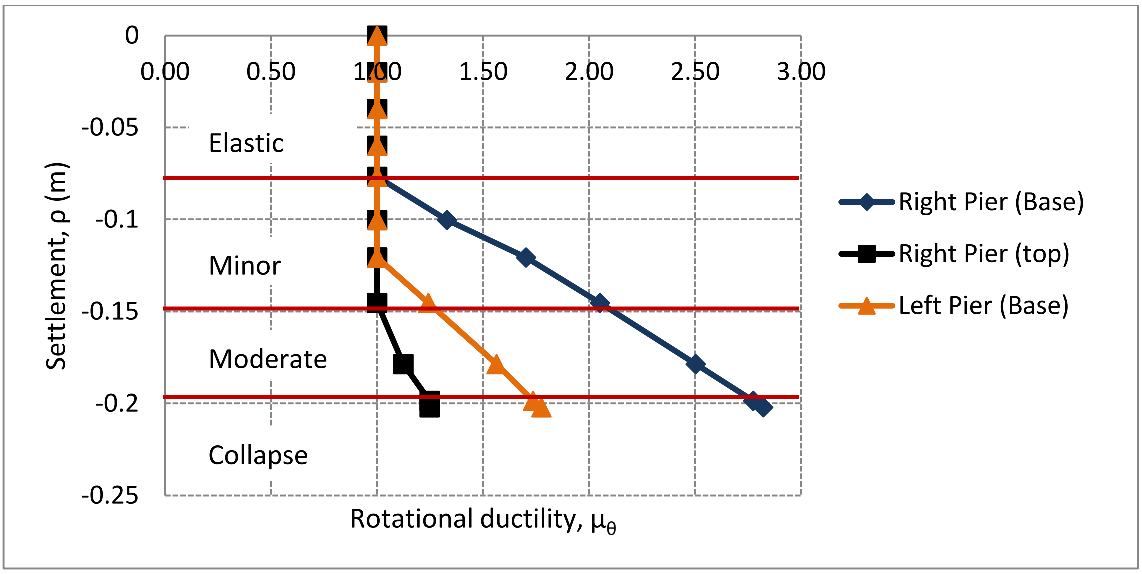


Figure 3.5. Applied settlements at the base of piers as a function of rotational ductility.

Σχήμα 3.5. Επιβαλλόμενες καθιζήσεις στη βάση των στύλων συναρτήσει της πλαστιμότητας στροφών.

4. ANNEX A

Analysis results obtained using SAP2000, wherein the lumped plasticity model was adopted for the NL behaviour of the piers were compared with those derived by nonlinear static analysis performed in OpenSees using fiber elements (distributed analysis). To this purpose the studied overpass was modelled in OpenSees. Elastic beam-column elements were used for the deck discretization, while the piers were modelled using non-linear beam-column fiber elements. The stress-strain relationships for the confined and the unconfined concrete were obtained from the Mander et al. (1988) model while the uniaxial Giuffré-Menegotto-Pinto material (Taucer et al., 1991) with isotropic strain hardening was used for the reinforcement bars. The median design strength of concrete and the yield strength of reinforcing steel are 27.5 and 500 MPa, respectively. Figure 3.6 presents a comparison of the bending moments at the base of the right pier as a function of the applied settlements derived from SAP2000 and OpenSees. The agreement is deemed satisfactory.

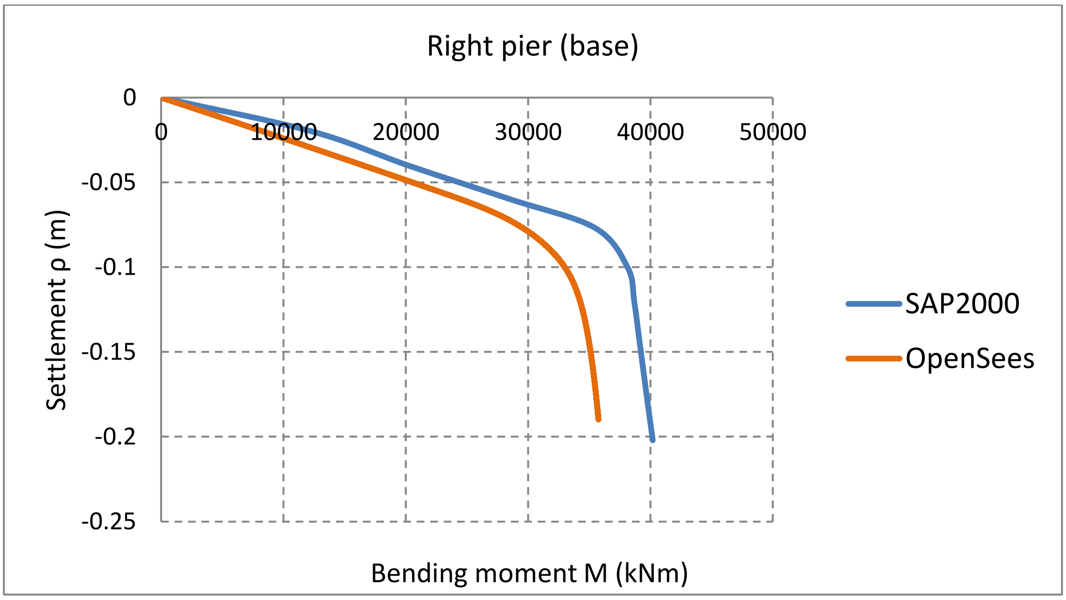


Figure 3.6. Comparison of lumped (SAP2000) and distributed (Opensees) models in the calculation of the tolerable settlement.

Σχήμα 3.6.Σύγκριση μοντέλων συγκεντρωμένης (SAP2000) και κατανεμημένης (Opensees) πλαστικότητας στον προσδιορισμό της επιτρεπόμενης καθίζησης.

5. References

Barker, R. M., Duncan, J. M., Rojiani, K. B., Ooi, P. S. K., Tan, C. K., and Kim, S. G. (1991). *Manuals for the Design of Bridge Foundations*, NCHRP Report 343, Transportation Research Board, National Research Council, Washington, DC.

Bozozuk, M. (1978). *Bridge foundations move*. TRB Research Record, (678), 17-21.

DiMillio, A. F. (1982). *Performance of Highway Bridge Abutments on Spread Footings on Compacted Fill*. Report No. FHWA RD-81-184, Federal Highway Administration, U.S. Department of Transportation.

Gifford, D. G., Kraemer, S. R., Wheeler, J. R. και McKown, A. F. (1987).*Spread Footings for Highway Bridges*. Report No. FHWA RD-86-185, Federal Highway Administration, U.S. Department of Transportation.

Grover, R.A. (1978).*Movements of bridge abutments and settlements of approach pavements in Ohio.* TRB Research Record, (678), 12-17.

Mander J, Priestley M, Park R. *Theoretical stress-strain model for confined concrete*. Journal of Structural Engineering, 1988; 114 (8): 1804–1826. DOI: 10.1061/(ASCE)0733-9445(1988)114:8(1804).

Moulton, L. K., GangaRao, H. V. S., Halvorsen, G. T. (1985). *Tolerable Movement Criteria for Highway Bridges*. Report No. FHWA RD-85-107, Federal Highway Administration, U.S. Department of Transportation.

Naresh, C., Samtani, N. C., Nowatzki, E. A. και Mertz, D. (2010).*Selection of Spread Footings on Soils to Support Highway Bridge Structures*. Report No. FHWA RD/TD-10-001, Federal Highway Administration, U.S. Department of Transportation.

Rossetto, T., Kappos, A.J., Kouris, L.A., Indirli, M., Borg, R.P., Lloyd, T.O., Sword-Daniels, V. (2010).Comparison of damage assessment methodologies for different natural hazards.*Proc. of COST Action C26 Final International Conference on Urban habitat construction under catastrophic events*, Naples.

Samtani, N. C. andNowatzki, E. A. (2006).*Soils and Foundations*.Vol. I και II, Report No. FHWA-NHI-06-088 and FHWA-NHI-06-089, Federal Highway Administration, U.S. Department of Transportation.

Sargand and Masada (2006).*Further Use of Spread Footing Foundations for Highway Bridges.* State Job Number 14747*(0)* Final Report FHWA-OH-2006/8 Final Report to Ohio Dept. of Transportation.

Stark, T.D., S.M.Olson, Long J.H. (1995).*Differential Movement at the Embankment/Structure Interface-Mitigation and Rehabilitation.* Report No. IAB-H1 FY93, Illinois DOT, Springfield, Illinois.

Taucer F, Spacone E, Filippou F. *A fiber beam-column element for seismic response analysis of reinforced concrete structures.* UCB/EERC-91/17, Earthquake Engineering Research Center College of Engineering, University of California, Berkeley, 1991.

Wahls, H. E. (1983). *Shallow Foundations for Highway Structures*. NCHRP Report 107, Transportation Research Board, National Research Council, Washington, D.C.

Wahls, H. E. (1990). *Synthesis of Highway Practice 159: Design and Construction of Bridge Approaches,* Transportation Research Board, National Research Council, Washington, D.C.

Walkinshaw, J.L. (1978). *Survey of bridge movements in the Western United States.* TRB Research Record, (678), 6-12.

Codes and Standards

AASHTO (2002).Standard Specifications for Highway Bridges.American Association of State Highway and Transportation Officials, Washington, D.C.

AASHTO (2007 with 2009 Interims). AASHTO LRFD Bridge Design Specifications. 4th Edition, American Association of State Highway and Transportation Officials, Washington, D.C.

EN 1990: 2002 Eurocode 0: Basis of Structural Design, European Committee for Standardization, Brussels.

EN 1992-2:2005. Design of concrete structures - Part 2: Concrete bridges.European Committee for Standardization, Brussels.

EN 1997-1:2004.Geotechnical design - Part 1: General rules.European Committee for Standardization, Brussels.

EN 1998-1:2004. Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings.European Committee for Standardization, Brussels.

EN 1998-2:2005. Design of structures for earthquake resistance - Part 2: Bridges. European Committee for Standardization, Brussels.

EN 1998-3:2005. Design of structures for earthquake resistance - Part 3: Assessment and retrofitting of buildings. European Committee for Standardization, Brussels.

EN 1998-5:2005. Design of structures for earthquake resistance - Part 5: Foundations, retaining structures and geotechnical aspects. European Committee for Standardization, Brussels.

FEMA-356 (2000). Prestandard and Commentary for the Seismic Rehabilitation of Buildings.ASCE, FEMA, Washington, D.C.