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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 6

Εφαρμογή σε Στατικώς Ορισμένη Γέφυρα Ο.Σ.

ΠΑΡΑΔΟΤΕΑ:

Κριτήρια Επιτελεστικότητας για Στατικώς Ορισμένη Γέφυρα Ο.Σ. (Π6α)

Ιανουάριος 2015

European Union
European Social Fund
OPERATIONAL PROGRAMME
EDUCATION AND LIFELONG LEARNING
investing in knowledge society
MINISTRY OF EDUCATION & RELIGIOUS AFFAIRS, CULTURE & SPORTS
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ΕΙΣΑΓΩΓΗ

Η παρούσα Τεχνική Έκθεση αποτελεί το **Παραδοτέο (Π6α)** 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 και με Συντονιστή της Ομάδας Εργασίας και Επιστημονικό Υπεύθυνο της Δράσης Δ6 τον Ιωάννη Ψυχάρη, Καθηγητή ΕΜΠ.

Συγκεκριμένα, παρουσιάζονται τα αποτελέσματα της Δράσης Δ6.1, με τίτλο:

"Εφαρμογή σε στατικώς Ορισμένη Γέφυρα από *Ο.Σ.*"

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

"Για ένα στατικώς ορισμένο σύστημα 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.

- *Μιάς εκτεταμένης μελέτης των διαθέσιμων σχετικών κανονισμών και οδηγιών (π.χ. Ευρωκώδικας 2 – Μέρος 2, Ευρωκώδικας 8 – Μέρος 2, Ευρωκώδικας 7, MCEER & FHA – κεφάλαιο 11.4),*
- *περιπτώσεων απόκρισης πραγματικών γεφυρών κατά τη διάρκεια πρόσφατων σεισμών, και*
- *παραμετρικών αναλύσεων διαφόρων δομικών στοιχείων της γέφυρας (π.χ. μεσοβάθρων, καταστρώματος, κλπ) υπό στατικές και δυναμικές φορτίσεις."*

ΜΕΘΟΔΟΛΟΓΙΑ ΚΑΙ ΑΠΟΤΕΛΕΣΜΑΤΑ

Μελετάται μία στατικώς ορισμένη γέφυρα Ο.Σ., κατά την επιβολή μετακινήσεων και στροφών στη βάση του μεσοβάθρου λόγω ρευστοποίησης των υποκείμενων εδαφικών σχηματισμών. Η γέφυρα θεωρείται πως θεμελιώνεται επιφανειακά σε έδαφος ρευστοποιήσιμο σε μεγάλο βάθος. Στόχος της διερεύνησης είναι ο προσδιορισμός των ανεκτών καθιζήσεων και στροφών που μπορεί να παραλάβει η γέφυρα χωρίς να αστοχήσει.

Η γέφυρα που μελετάται αποτελείται από δύο αμφιέρειστα ανοίγματα θεωρητικού μήκους 42.00m το καθένα, τα οποία συνδέονται μεταξύ τους με πλάκα συνέχειας. Το πλάτος του καταστρώματος ισούται με 11.25m με δύο πεζοδρόμια εκατέρωθεν πλάτους 1.25μ έκαστο. Αποτελείται από 14 (=2x7) προκατασκευασμένες, προεντεταμένες δοκούς μήκους 40.50m η κάθε μια, που στηρίζονται στα ακρόβαθρα και τα μεσόβαθρα μέσω ελαστομεταλλικών εφεδράνων. Το μεσόβαθρο αποτελείται από τη δοκό έδρασης και ένα στύλο τοιχοειδούς διατομής μήκους 8.35m και πάχους 1.5m από οπλισμένο σκυρόδεμα, έχει δε ύψος 10.0m συμπεριλαμβανομένης της δοκού έδρασης (8.0+2.0m).

Σε στατικώς ορισμένες γέφυρες, το πιο κρίσιμο δομικό στοιχείο για την ανοχή του συνόλου της γέφυρας σε επιβαλλόμενες μετακινήσεις λόγω ρευστοποίησης, είναι το μεσόβαθρο καθώς η ανάπτυξη πλαστικής άρθρωσης στη βάση του, οδηγεί σε μετατροπή του συνολικού στατικού συστήματος σε μηχανισμό που δεν μπορεί να φέρει ούτε καν τα κατακόρυφα φορτία. Αστοχία σε δευτερεύοντα στοιχεία (όπως εφέδρανα, πλάκα συνέχειας ή αρμοί) που δεν μπορούν να προκαλέσουν κατάρρευση της γέφυρας ενώ μπορούν να επισκευαστούν ή και να αντικατασταθούν εύκολα, δεν λαμβάνονται υπόψη για τον καθορισμό των επιτρεπόμενων εδαφικών μετακινήσεων.

Μέσω προσομοίωσης της γέφυρας με γραμμικά πεπερασμένα στοιχεία επιβάλλονται σταδιακά κατακόρυφες μετακινήσεις και στροφές στο επίπεδο θεμελίωσης του μεσοβάθρου και εκπονούνται μη γραμμικές αναλύσεις με χρήση του λογισμικού Opensees, λαμβάνοντας υπόψη τη γεωμετρική μη γραμμικότητα, καθώς και τη μη γραμμικότητα των υλικών. Ένα προσομοίωμα κατανεμημένης πλαστιμότητας λαμβάνεται υπόψη για τη μη γραμμική συμπεριφορά του στύλου, η οποία περιγράφεται με τη χρήση διαγραμμάτων ροπών–καμπυλοτήτων. Επί των καμπυλών αυτών, κατόπιν κατάλληλης διγραμμικοποίησης, ορίζεται η ροπή διαρροής M_{Rd} κατά την οποία αναπτύσσεται για πρώτη φορά πλαστική άρθρωση στη βάση του στύλου.

Λαμβάνοντας υπόψη ότι οι καθιζήσεις και οι στροφές λόγω ρευστοποίησης συσσωρεύονται σταδιακά κατά την διάρκεια της δόνησης και λαμβάνουν την μέγιστη τιμή τους στο τέλος της, οι επιβαλλόμενες μετακινήσεις Δ εφαρμόζονται στην κατασκευή ως μόνιμα φορτία υπό τους ακόλουθους στατικούς συνδυασμούς: $1.15G+1.15\Delta+P+1.35Q$ και $G+1.15\Delta+P$. Οι καθιζήσεις θεωρούνται τμήμα των μόνιμων δράσεων της κατασκευής μετά το σεισμό και λαμβάνουν τη μορφή $\Delta = \rho + \varphi_y + 0.3\varphi_x$ ή $\Delta = \rho + 0.3\varphi_y + \varphi_x$ όπου οι στροφές φ_x και φ_y περί τον ισχυρό και τον ασθενή άξονα του βάθρου αντίστοιχα, συνδέονται άμεσα με την καθίζηση ρ με την εμπειρική σχέση $\varphi_x = \varphi_y = 0.05\rho$

Από τις αναλύσεις προέκυψε πως κρισιμότερος είναι ο συνδυασμός που περιλαμβάνει στροφή περί τον ασθενή άξονα του βάθρου. Διαπιστώθηκε ότι το βάθρο συνεχίζει να λειτουργεί πρακτικά ελαστικά έως καθίζηση ρ της τάξεως των 23cm, ενώ η συνδυασμένη στροφή ως προς ασθενή άξονα του βάθρου ανέρχεται τότε σε 0.020rad και η αντίστοιχη γωνία κλίσης του βάθρου σε 1.10%.

Τέλος, εξετάστηκε και μια πιο απλοποιημένη μέθοδος, όπου η καμπύλη ροπών-καμπυλοτήτων στη βάση του στύλου υπολογίζεται με βάση τις απλοποιημένες σχέσεις του ΚΑΝΕΠΕ, παράρτημα 7Α. Με δεδομένες πια, τη ροπή διαρροής του στύλου $M_{Rd,y}$, και την αρχική ελαστική ακαμψία του, αρκεί να εκτελεστεί μόνον μια γραμμική ελαστική ανάλυση της γέφυρας για μια τυχαία τιμή καθίζησης ρ_{act}. Εάν M_{act} είναι η καμπτική ροπή στη βάση του στύλου που αντιστοιχεί σε αυτή τη τιμή της καθίζησης, η μέγιστη επιτρεπόμενη καθίζηση ρ_{all} δίνεται από τη σχέση:

$\rho_{all} = \rho_{act} \times (M_{Rd,y} / M_{act})$

Διαπιστώθηκε ότι τα αποτελέσματα της απλοποιημένης μεθόδου είναι παρόμοια, αν και πιο συντηρητικά, από εκείνα της προηγούμενης μεθόδου. Συγκεκριμένα, το βάθρο συνεχίζει να λειτουργεί πρακτικά ελαστικά έως καθίζηση ρ της τάξεως των 18cm, ενώ η συνδυασμένη στροφή ως προς ασθενή άξονα του βάθρου ανέρχεται τότε σε 0.016rad και η αντίστοιχη γωνία κλίσης του βάθρου σε 1.09%.

Συμπερασματικά φαίνεται ότι για στατικώς ορισμένες γέφυρες, καθιζήσεις της τάξεως των 23cm που εισάγονται στη θεμελίωση λόγω ρευστοποίησης, μπορεί να γίνουν ανεκτές προκαλώντας μόνον αμελητέες βλάβες στο βάθρο, το οποίο πρακτικά συνεχίζει να λειτουργεί ελαστικά. Αυτές οι τιμές φαίνεται να είναι συμβατές με εκείνες που προτείνονται στη βιβλιογραφία για την οριακή κατάσταση αστοχίας.

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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 6

Application to Statically Determinate Concrete Bridge

DELIVERABLES

Performance Criteria
for Statically Determinate R.C. Bridges (D6a)

January 2015

European Union
European Social Fund
OPERATIONAL PROGRAMME
EDUCATION AND LIFELONG LEARNING
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This Technical Report (**D6a**) presents the actions taken and the associated results of the first part of the Work Package WP6, entitled: *"Application to statically determinate concrete bridges"*.

The Scope of **Work Package WP6** 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 statically determinate concrete bridges, which probably consist the most common type of bridge in our country.

The main work tasks required to achieve the aim of this first part of the Work Package (WP6.1) are the following:

"For a statically determinate concrete bridge, the allowable foundation movements (settlements and rotations) will have to be established. The relevant criteria will take into account the permissible damage and serviceability levels (e.g. driving discomfort, repairable damage, non-repairable damage), as well as the anticipated seismicity level (e.g. seismic excitation with 90, 450 or 900 years return period), and will be established after 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 analyses of various bridge components (e.g. piers, deck, etc.) under static and cyclic dynamic loading.*

Table of contents

1. Introduction

The decisive factor for designing bridges with shallow foundations over liquefaction susceptible soils is the magnitude of the additional ground movements <u>permanently</u> induced to the bridge footings after soil liquefaction in the form of vertical displacements (settlements) and rotations. This additional liquefaction-induced displacements may produce additional stress and deformations to the structural components of the superstructure (piers, deck, bearings, etc.) and potentially contribute to the loss of its serviceability and its structural integrity. To this end, the determination of the tolerable ground deformations to the superstructure is a necessary and critical prerequisite for the design of such a foundation.

Unfortunately, contemporary seismic codes and guidelines do not provide specific limits for foundation deformations, both during and after seismic excitation. Because of the large number of factors involved (structural system, materials, soil type, intended use and life of the structure, etc.) only approximate values of tolerable settlements under static conditions are given in the international literature, usually solely depending on the bridge structural configuration. An extensive literature survey, as well as the provisions of various Codes are explicitly presented in chapter 2.

In chapters 3 and 4, for the statically determinate concrete bridge examined within the frame of WP6, tolerable ground deformations (base settlements and rotations) are defined, as the first step of the innovative design process. Limiting values are defined on the basis of the structural integrity of the structure. Then, in chapter 5 a review of the allowable ground deformations values concerning serviceability of the bridge is made according to the information supplied in chapter 2.

2. Literature survey and codes provisions

In the following, a literature review as well as the provisions of contemporary seismic codes and guidelines are 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 2.1, 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 in which 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.

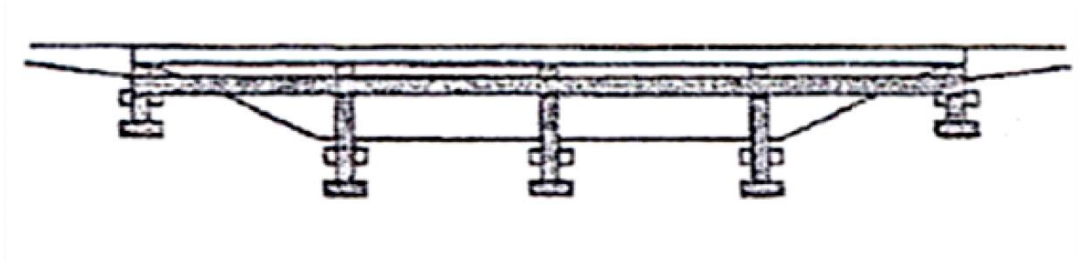
Non- uniform settlements, when the settlement at each support of a multi span bridge is different. It may be either regular or irregular as noted in Figures 1(c) & 1(d). A regular pattern in deformation is characterized by a symmetrical distribution of settlement, from both ends of the bridge towards 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:

$$\beta=\frac{\delta}{S}$$

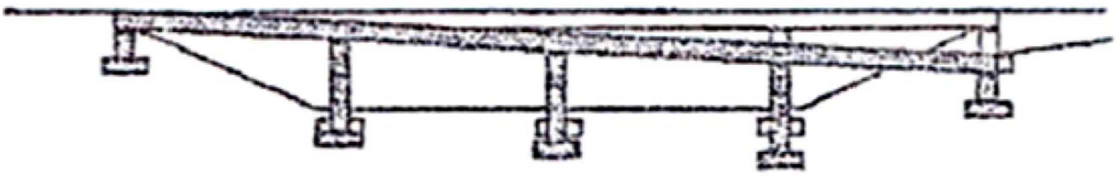
where,

β = angular distortion (dimensionless)

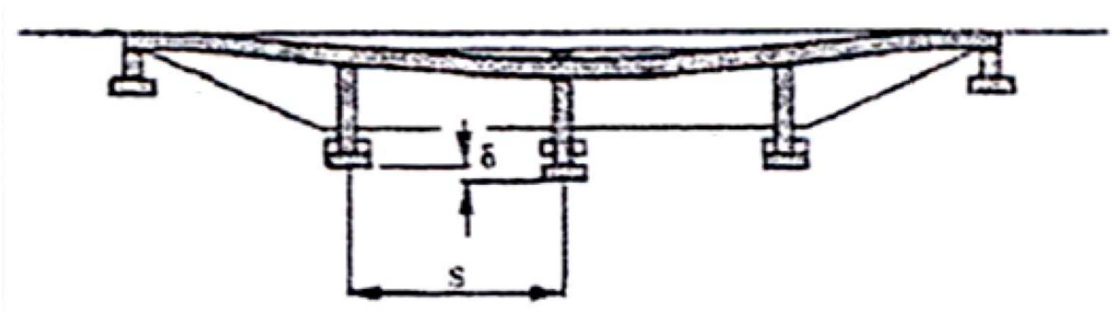
δ =differential settlement between two consecutive foundations; in units of length



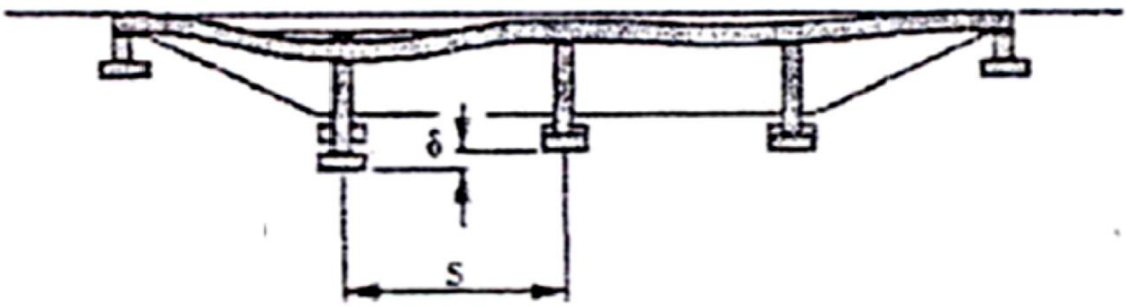
(a) Uniform Settlement (ρ)



(b) Uniform tilt (ω) or rotation (θ)



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



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

Figure 2.1: 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)

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

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 rockers, 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. Neoprene 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 floor 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 relief joints may also be necessary (Moulton et al., 1985).

2.2. Movement Criteria

The selection of limiting values of imposed displacements consists 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, etc.

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 for 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".*

2.2.1. Literature survey

In the following, a literature overview is attempted 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 on 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, without specific distinction in terms of type or size. 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 parallel to that of Walkinshaw (1978) and Grover (1978) and was documented via an extensive research on allowable displacements undertaken in the U.S.A. and Canada and published by the Transportations 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 2.1).

DiMillio (1982) attempted to evaluate the behavior 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 satisfactory foundation soils.

Moulton et al. (1985) carried out a survey that was based on a nationwide study of 314 concrete and steel bridges on spread footings in 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 Transportation Research Board's Committee A2K03 was adopted. The results were classified according to the type of spans, to the length and stiffness of spans and to the type of construction material. It was shown that many highway bridges can tolerate significant magnitudes of total and differential vertical settlement without becoming seriously overstressed, sustaining serious structural damage, or suffering impaired riding quality. In particular, it was found that a longitudinal angular distortion (differential settlement to span length) 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 2.1: 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)

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. In addition differential movements not greater than 2 inches (50 mm) laterally and less than 4 inches (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, classified in increasing order of magnitude. In Table 2.2, proposed serviceability criteria for bridges by the aforementioned researchers are summarized.

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) steel	Moulton et al. (1985)
	0.005 (1/200)	Simply supported steel/concrete bridges with l ≥ 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 & concrete bridges)	Moulton et al. (1985)
	< 31.75	Bridge pier following bridge completion (steel bridges)	Moulton et al. (1985)
	< 38.1	Bridge pier following bridge completion (concrete bridges)	Moulton et al. (1985)
Horizontal displacements (mm)	< 38	Acceptable	Moulton et al. (1985)
Horizontal along with vertical displacements (mm)	< 25	Acceptable	Moulton et al. (1985)

2.2.2. 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 to limiting values of measurable deformations either of the structure or the foundation (performance levels). However, the main attitude of the Codes is that the desired behavior 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 behavior of a bridge under static and dynamic actions is today indirectly fulfilled, when the structure satisfies two limit states, the Ultimate Limit State (ULS) and the Serviceability Limit State (SLS).

Ultimate Limit State (ULS) is associated with the safety of the 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), deals with 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 operational efficiency of the structure and finally damage that affect 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 analyses. 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 , wingwalls, 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 EN1992-2:2005, the effects of uneven settlements of the structure due to soil subsidence should be considered for the verification for 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 behavior 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 it is 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 given 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 from 1/2000 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. (see 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 behavior 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 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 no 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 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 untouched; 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 behavior 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.

2.2.3. Other approaches

On the other hand, other approaches may be adopted to specify limiting values for foundation movements of bridges.

According to 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.

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 2.4: 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

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. Estimation of tolerable settlements and rotations

3.1. Methodology

As a first step, a preliminary design against static and seismic actions should be carried out, in order to initially define the geometrical characteristics and the required reinforcement of the various structural members. In this analysis, the results of the conventional design are used, for comparative reasons (see Deliverable D6b). The maximum allowable settlement ρ_{all} of the pier is calculated by performing appropriate analyses as described in the following.

It is reminded that the bridge under investigation is a statically determinate, two-span (2×42.00m) concrete bridge. The deck is 11.25m wide (with 1.25m pavement at each side), composed of 2×7 precast, prestressed concrete beams of length 40.50m. A cast in-situ slab of 0.25m min thickness is constructed. The concrete beams are resting upon the abutments and the mid-pier via elastomeric bearings. The pier is a wall-type column, 8.0m high, 1.5 m thick and 8.35 m wide, founded on a soil prone to liquefaction under the design seismic action. The geometrical characteristics of the bridge and the reinforcement of the pier calculated from the conventional design are presented in Figures 3.1 to 3.3.

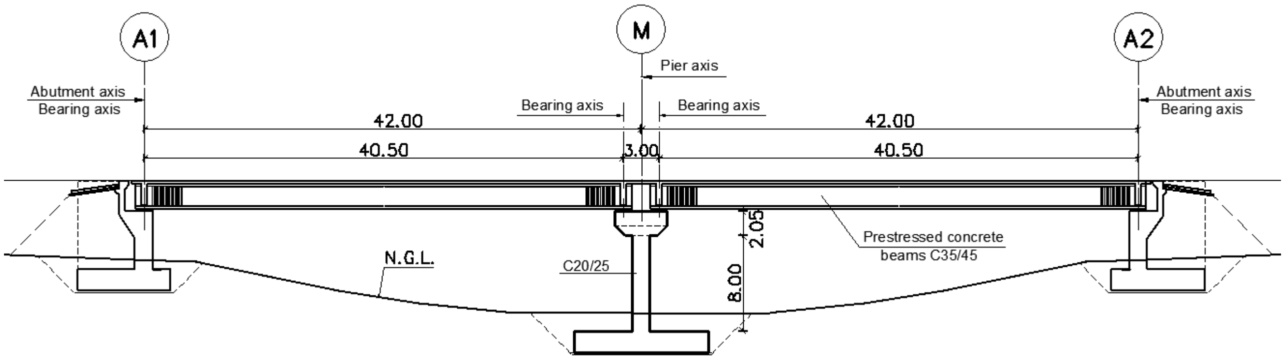
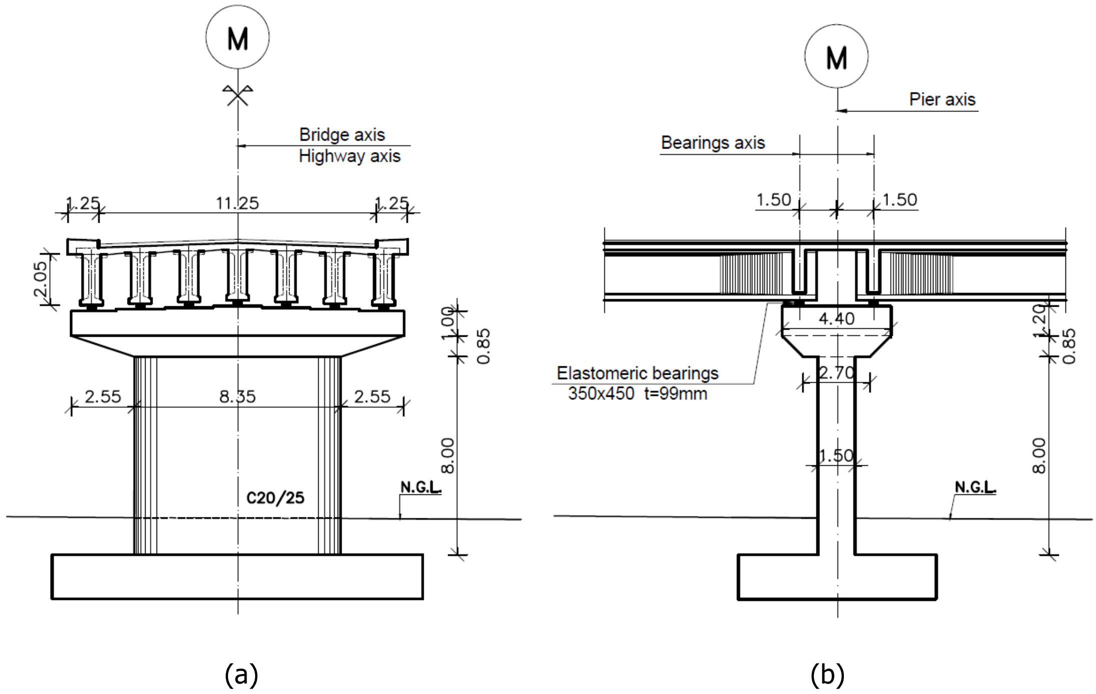


Figure 3.1: Londitudinal section of the bridge

Σχήμα 3.1: Κατά μήκος τομή γέφυρας



(a) (b)

Figure 3.2: Geometry of the pier : (a) in longitudinal direction , (b) in lateral direction of the bridge

Σχήμα 3.2: Γεωμετρία μεσοβάθρου: (a) στη διαμήκη διεύθυνση, (b) στην εγκάρσια διεύθυνση της γέφυρας

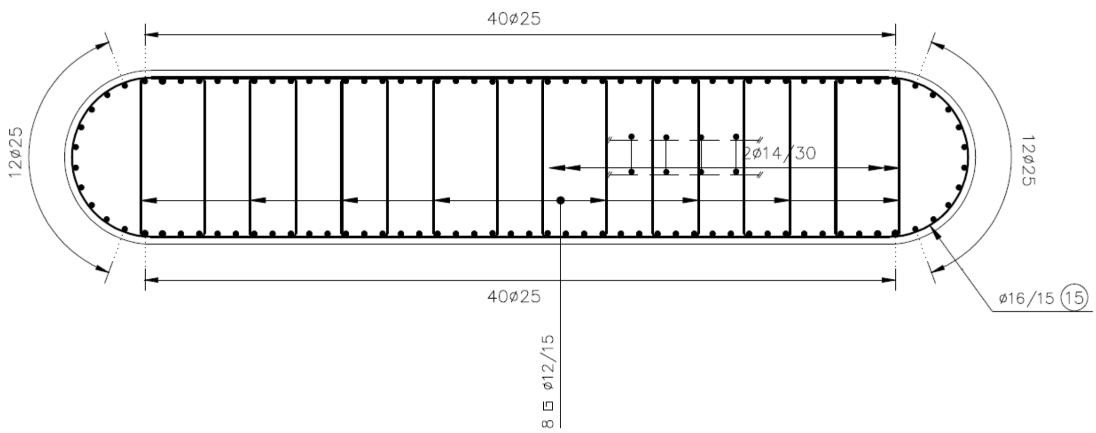


Figure 3.3: Pier reinforcement (base section)

Σχήμα 3.3: Οπλισμός μεσοβάθρου (διατομή βάσης)

3.2. Loads and load combinations

It is known that liquefaction-produced displacements (in the form of settlements and rotations) gradually accumulate during vibration receiving their maximum value at the end of the seismic event (Karamitros et al. 2012, Karamitros et al. 2013a and Karamitros et al. 2013b). Therefore, these displacements should be considered as **permanent** actions, which may produce permanent additional stress and deformations to the structural components of the superstructure (piers, deck, bearings, etc.).

The load combination proposed by EC0 for persistent design situations (equation 6.10b) should be examined:

$$\xi \cdot \gamma_G \cdot G \text{ “+” } \gamma_P \cdot P \text{ “+” } \gamma_Q \cdot Q \qquad (1)$$

where, the reduction factor ξ for unfavorable permanent actions is equal to 0.85 and the partial factors for permanent, prestressing and variable (traffic loads) actions are taken as $\gamma_G=1.35$, $\gamma_P=1.0$ and $\gamma_Q=1.35$ respectively. In the aforementioned relationship, settlements and rotations are considered as permanent loads. It is:

$$G = G_{DL+SDL} + \Delta \qquad (2)$$

where G_{DL+SDL} are the dead and super dead loads of the structure and Δ the most unfavorable of the next combinations:

$$\Delta = \rho + \varphi_y + 0.3\varphi_x \qquad (3), \text{ or}$$

$$\Delta = \rho + \varphi_x + 0.3\varphi_y \qquad (4)$$

Moreover, settlements and rotations are considered to be related according to the following empirical relationship, though derived for buildings (Yasuda, 2014):

$$\varphi_x = \varphi_y = 0.05 \cdot \rho \qquad (5)$$

where: ρ is the induced settlement (in cm) and φ_x, φ_y the resultant rotations about the strong and the weak axis of the pier, respectively (in degrees).

3.3. Movement criteria

For statically determinate bridges the most critical structural member is the pier, which essentially dictates the tolerance of the entire system to liquefaction-induced deformations. In such types of structural systems, where the piers are simple cantilevers, the formation of plastic hinges at their base is not allowed, since the structural system becomes a mechanism unable to carry even the vertical loads (see Figure 3.4).

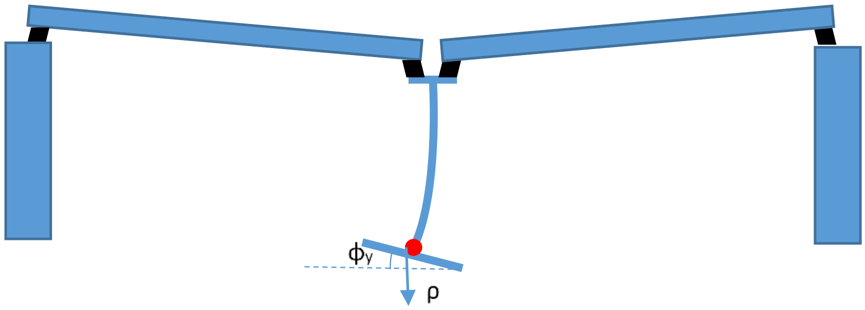


Figure 3.4: A statically determinate bridge is converted to a mechanism after the formation of a plastic hinge at the base of the pier

Σχήμα 3.4: Οι ισοστατικές γέφυρες μεταπίπτουν σε μηχανισμούς μετά την δημιουργία πλαστικής άρθρωσης στη βάση του μεσοβάθρου

As a consequence, the maximum allowable settlement ρ_{all} of the pier, for the load combinations mentioned in 2.1, is set equal to the settlement which produces moment at the base of the pier equal to the yielding moment M_y. In this way, after liquefaction, damage of the pier will be avoided. The fact that no damage of the piers is allowed is completely compatible with the conventional design of this type of bridges against seismic actions, where a practically elastic behavior of the piers is required ($q \leq 1.5$). Nevertheless, it should be noted that the capacity of the pier to carry static and future seismic loads will be reduced, as a result of the additional stress developed, mainly due to the imposed rotations at its base.
Other, secondary structural members that can easily be repaired or replaced (e.g. bearings, joints etc.) are allowed to suffer more serious damage, so they are not taken into account in the calculation of ρ_{all}.

3.4. Analysis procedure

Firstly, the yield bending moment $M_{Rd,y}$ of the bottom section of the pier is calculated. To this end, the bending moment – curvature curve of this section is drawn, using the software Opensees.

Two load combinations are examined, in order to account for the maximum and the minimum axial load of the pier, namely:

for N_{max} :

$$\xi \cdot \gamma_G \cdot (G+\Delta) \text{ “+” } \gamma_P \cdot P \text{ “+” } \gamma_Q \cdot Q \;=>\; (0.85\text{x}1.35) \cdot (G+\Delta) \text{ “+” } P \text{ “+” } 1.35Q \;=>$$

$$1.15G + 1.15\Delta \text{ “+” } P \text{ “+” } 1.35Q \qquad (6)$$

for N_{min} :

$$G + \xi \cdot \gamma_G \cdot \Delta \text{ “+” } P \;=>\; G + 1.15\Delta \text{ “+” } P \qquad (7)$$

Taking under consideration that the expected results are to be used for design reasons, design values are used for the properties of the materials. The following properties were considered for concrete type C20/25 and steel bars S500s according to EC2:

$$f_{cd}=\frac{\alpha_{cc}f_{ck}}{\gamma_c}=\frac{0.85\cdot 20}{1.5}=11.33MPa$$

$\varepsilon_{c0}=0.002$

$\varepsilon_{cu}=0.0035$

and

$$f_{yd}=\frac{f_{yk}}{\gamma_s}=\frac{500}{1.15}=434.78MPa$$

$E_s=200GPa$

$\varepsilon_{ud}=0.9\varepsilon_{uk}=0.9x0.075=0.0675$

The moment – curvature curve at the base of the pier is illustrated in Figure 3.5 and 3.6 for N_{max} and N_{min} respectively. The bilinear approximation of the original curves is also shown. The rule of almost equal areas above and below the actual curve is used, along with the fact that the initial branch of the bilinear curve passes from that point of the actual curve which corresponds to 60% of the resulting yield moment $M_{Rd,y}$.

The results are summarized in Table 3.1, where the yield moment at the base of the pier $M_{Rd,y}$, the corresponding curvature $(1/r)_y$, the failure moment $M_{Rd,u}$ and the corresponding curvature $(1/r)_u$ are given. Also k_e is the initial elastic stiffness and K_p the stiffness of the plastic branch.

Table 3. 1: Bilinear approximations of moment-curvature curves at the base of the pier

Πίνακας 3.1: Διγραμμικές προσεγγίσεις του διαγράμματος ροπών-καμπυλοτήτων στη βάση του μεσοβάθρου

	$M_{Rd,y}$ (kNm)	**$(1/r)_y$** (m^{-1})	**$M_{Rd,u}$** (kNm)	**$(1/r)_u$** (m^{-1})	**$\alpha=K_p/k_e$**
N_{max}	28275	2.05×10^{-3}	29072	9.80×10^{-3}	0.725%
N_{min}	25850	2.21×10^{-3}	26320	14.59×10^{-3}	0.325%

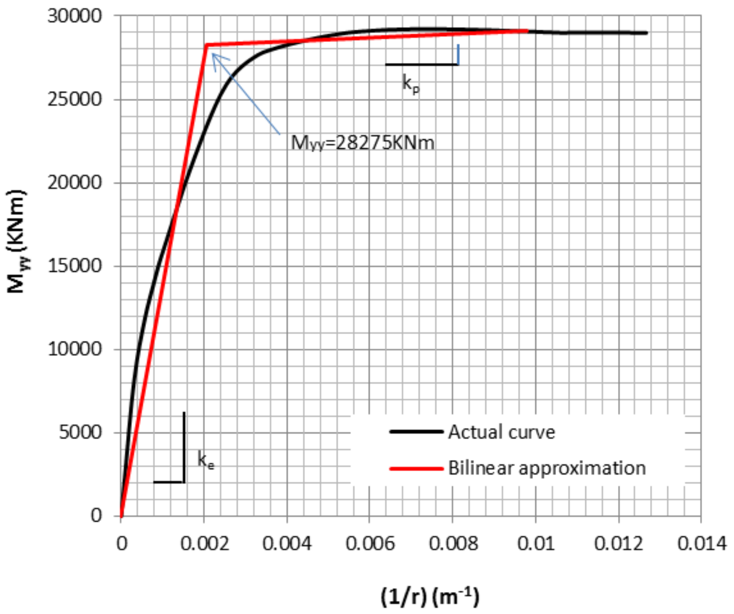


Figure 3.5: Moment- curvature curve at the base of the pier and its bilinear approximation (for Nmax=25131KN)

Σχήμα 3.5: Διάγραμμα ροπών-καμπυλοτήτων στη βάση του μεσοβάθρου και η διγραμμική του προσέγγιση (για Nmax=25131KN)

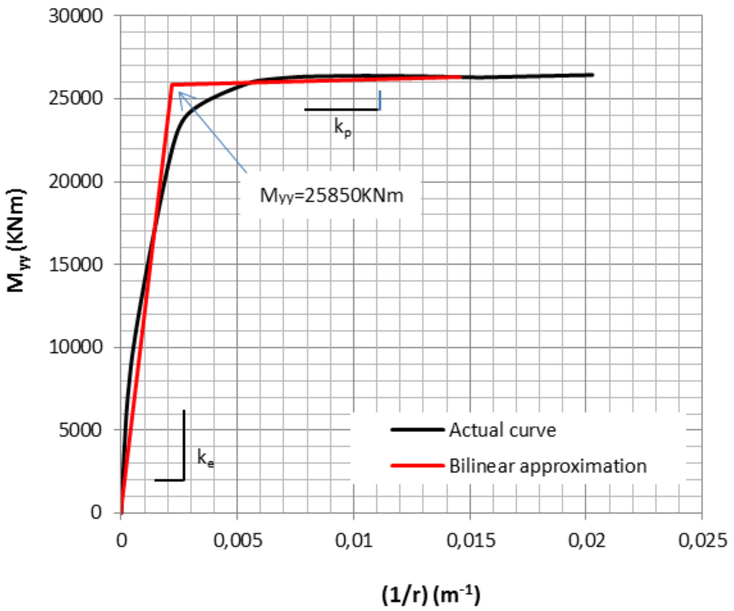


Figure 3.6: Moment- curvature curve at the base of the pier and its bilinear approximation (for Nmin=18596KN)

Σχήμα 3.6: Διάγραμμα ροπών-καμπυλοτήτων στη βάση του μεσοβάθρου και η διγραμμική του προσέγγιση (για Nmin=18596KN)

Then, a refined finite element model of the studied bridge is prepared (see Figure 3.7). The maximum allowable settlement ρ_{all} of the pier is calculated, through the software Opensees, as the settlement at which the yield moment at the base of the pier is reached, under the persistent combinations of loads, presented in equations (6) and (7). To this end, a nonlinear static (pushover) analysis is performed through a step-wise procedure of gradually increasing settlements and rotations at the base of the pier, according to equations (3) to (5). A distributed plasticity model is adopted for the non-linear behaviour of the pier.

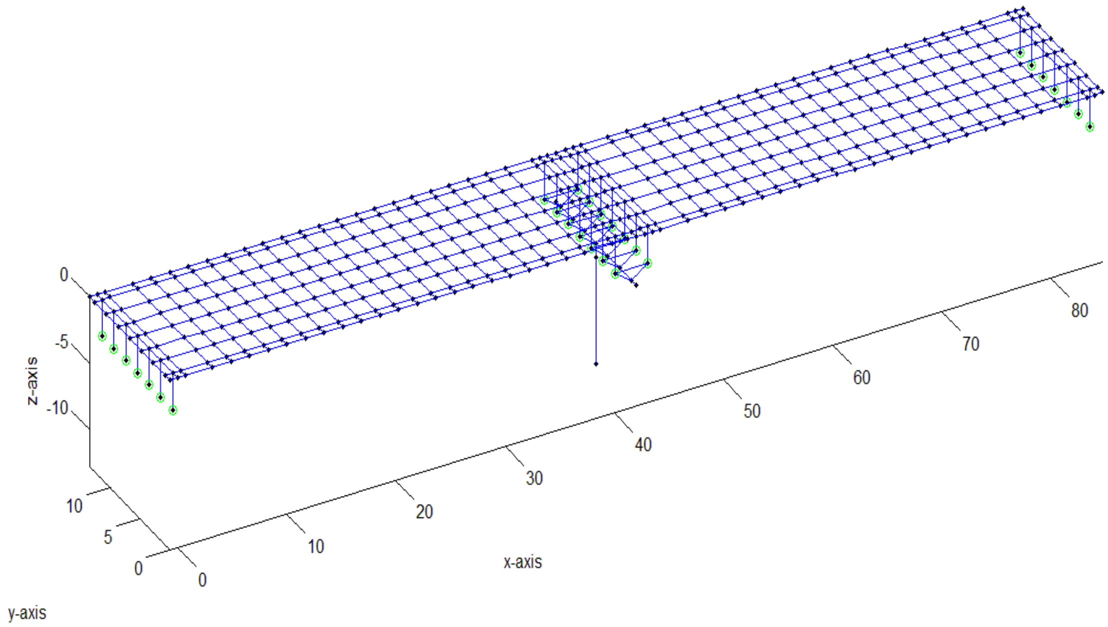


Figure 3.7: Finite element model of the bridge by means of the software Opensees

Σχήμα 3.7: Προσομοίωμα πεπερασμένων στοιχείων της γέφυρας με χρήση του λογισμικού Opensees

It was found that the most critical combination of imposed displacements Δ, is that of equation (3), namely $\Delta = \rho + \varphi_y + 0.3\varphi_x$, mainly due to the imposed rotation φ_y about the weak axis of the section.

In Figures 3.8 and 3.10 the moment M_{yy} developed about the weak axis of the pier is computed against the imposed settlement at the base of the pier, for N_{max} and N_{min} respectively. Then, the allowable settlement is defined as the critical settlement producing the yield moment $M_{Rd,y}$ at the base of the pier.

The corresponding inclination angles (drifts) θ of the pier, defined as θ=δ/h, are illustrated in Figures 3.9 and 3.11 for N_{max} and N_{min} respectively, where δ the relative

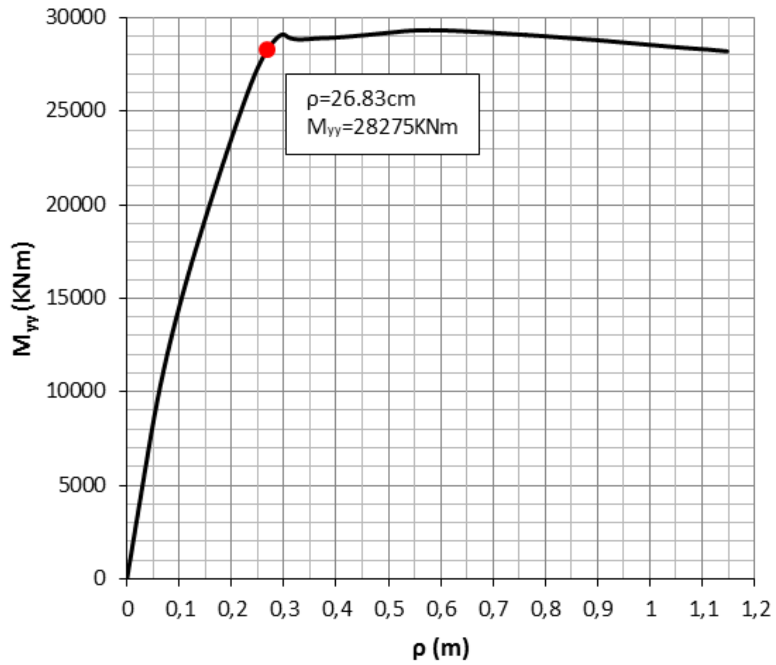


Figure 3.8: Moments at the base of the pier against imposed settlements derived from pushover analysis (for N_{max}=25131KN)

Σχήμα 3.8: Διάγραμμα ροπών στη βάση του μεσοβάθρου έναντι επιβαλλόμενων καθιζήσεων, όπως προέκυψε από τη μη γραμμική ανάλυση (για N_{max}=25131KN)

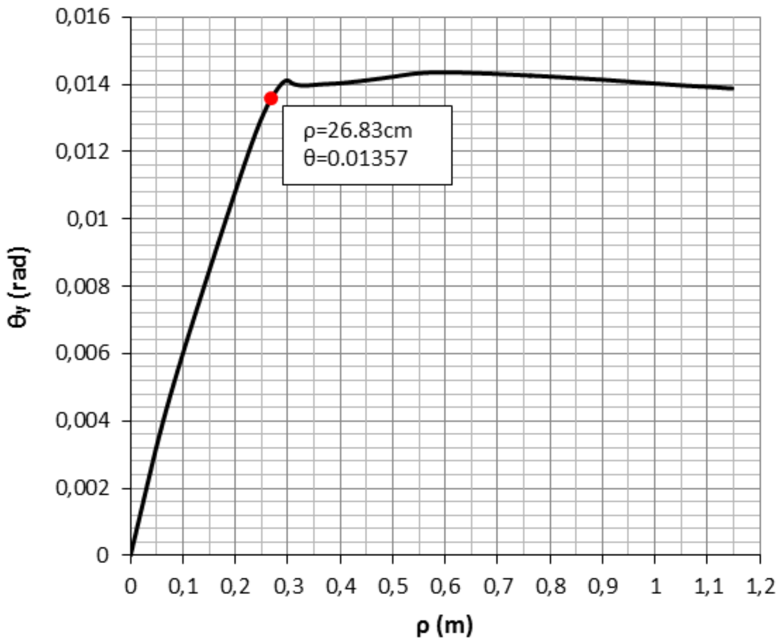


Figure 3.9: Inclination angle of the pier against imposed settlement derived from pushover analysis (for N_{max}=25131KN)

Σχήμα 3.9: Γωνία κλίσης μεσοβάθρου έναντι επιβαλλόμενων καθιζήσεων όπως προέκυψε από τη μη γραμμική ανάλυση (για N_{max}=25131KN)

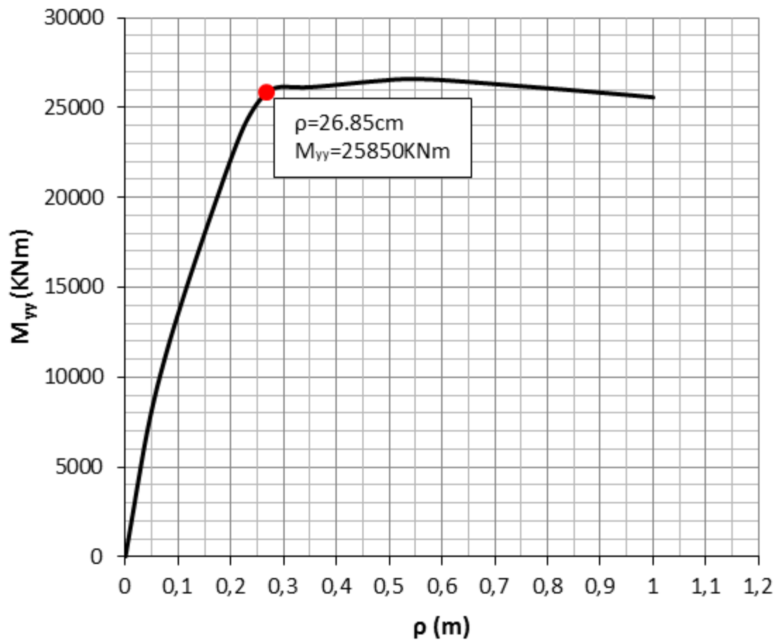


Figure 3.10: Moment at the base of the pier against imposed settlement derived from pushover analysis (for N_{min}=18596KN)

Σχήμα 3.10: Διάγραμμα ροπών στη βάση του μεσοβάθρου έναντι επιβαλλόμενων καθιζήσεων όπως προέκυψε από τη μη γραμμική ανάλυση (για N_{min}=18596KN)

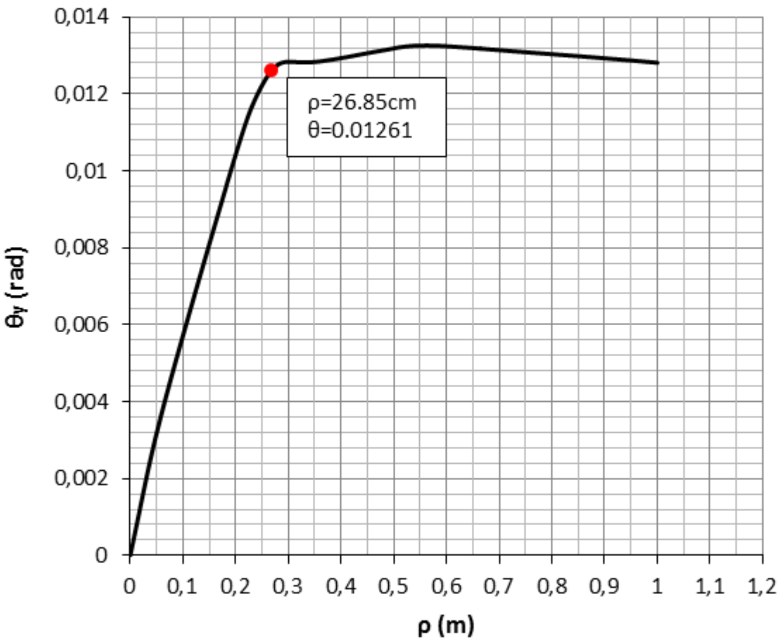


Figure 3.11: Inclination angle of the pier against imposed settlement derived from pushover analysis (for N_{min}=18596KN)

Σχήμα 3.11: Γωνία κλίσης μεσοβάθρου έναντι επιβαλλόμενων καθιζήσεων όπως προέκυψε από τη μη γραμμική ανάλυση (για N_{min}=18596KN)

displacement between the top and the bottom of the pier and h the height of the pier.

The allowable values of the critical parameters for the design of the innovative bridge, according to the aforementioned results are summarized in Table 3.2. It should be noted that the results are divided by the safety factor of 1.15 which has been taken into account in Equations (6) and (7).

As a conclusion, for the most unfavorable loading case of N_{min}, the allowable settlement is found equal to **23.3 cm,** corresponding to an inclination angle of the pier equal to **1.10%.**

Table 3.2: Allowable values of critical parameters for the design of the innovative bridge derived from the non-linear analysis

Πίνακας 3.2: Μέγιστες επιτρεπτές τιμές κρίσιμων παραμέτρων για το σχεδιασμό της πρωτότυπης γέφυρας, που προέκυψαν από τη μη γραμμική ανάλυση

Combination	ρ_{all} (cm)	$\varphi_{y,y}$ (rad)	$\varphi_{x,y}$ (rad)	θ_y (rad)	$M_{Rd,y}$ **(kNm)**
1.15(G+S)+P+1.35Q (for N_{max})	26.83/1.15= **23.3**	**0.020**	**0.006**	0.01357/1.15= **0.0118**	**28275**
G+1.15S+P (for N_{min})	26.85/1.15= **23.3**	**0.020**	**0.006**	0.01261/1.15= **0.0110**	**25850**

4. A simplified approach

A more simplified method could be adopted for design purposes. First, the moment – curvature curve is obtained for the bottom section of the pier by means of the relationships proposed in Annex 7A of the Greek Code for Interventions (KANEPE). Then, given the initial elastic stiffness of the pier and its yield moment $M_{Rd,y}$, a linear elastic analysis of the bridge is performed for a random value of settlement ρ_{act} and for the two load combinations expressed by equation (6) and (7). Only a single elastic analysis is needed, through which the bending moment M_{act} at the bottom of the pier, which corresponds to the random value of settlement ρ_{act}, is defined. The max allowable value of settlement ρ_{all} is then calculated as:

$\rho_{all} = \rho_{act} \times (M_{Rd,y} / M_{act})$

4.1. Calculation of yield curvature of the base section (Annex 7A of KANEPE)

I) Yield curvature due to tension reinforcement yielding

$$\left(1/r\right)_y = \frac{f_y}{E_s \cdot (1-\xi_y) \cdot d}$$

where:

$$\xi_y = (\alpha^2 \cdot A^2 + 2 \cdot \alpha \cdot B)^{1/2} - \alpha \cdot A$$

and

$$\alpha = \frac{E_s}{E_c},$$

$$A = \rho + \rho' + \rho_v + \frac{N}{b \cdot d \cdot f_y},$$

$$B = \rho + \rho' \cdot \delta' + 0.5 \cdot \rho_v \cdot (1+\delta') + \frac{N}{b \cdot d \cdot f_y} \quad \text{and}$$

$$\delta' = \frac{d'}{d}$$

II) Yield curvature due to concrete deformations

$$\left(1/r\right)_y = \frac{1.8 \cdot f_c}{E_c \cdot \xi_y \cdot d}$$

where:

$$\xi_y = (\alpha^2 \cdot A^2 + 2 \cdot \alpha \cdot B)^{1/2} - \alpha \cdot A$$

and

$$\alpha = \frac{E_s}{E_c},$$

$$A = \rho + \rho' + \rho_v - \frac{N}{1.8 \cdot \alpha \cdot b \cdot d \cdot f_c},$$

$$B = \rho + \rho' \cdot \delta' + 0.5 \cdot \rho_v \cdot (1 + \delta') \quad \text{and}$$

$$\delta' = \frac{d'}{d}$$

Also:

d and d′ are the depths to the tension and compression reinforcement, respectively.

$$d = h - c - d_w - \frac{d_b}{2} \quad \text{and}$$

$$d' = c + d_w + \frac{d_b}{2}$$

$\rho = \frac{A_s}{b \cdot d}$ is the tension reinforcement ratio and A_s is the tension reinforcement area,

$\rho' = \frac{A_s'}{b \cdot d}$ is the compression reinforcement ratio and A_s' is the compression reinforcement area,

$\rho_v = \frac{A_v}{b \cdot d}$ ratio of longitudinal reinforcement between compressed and tensioned fiber and A_v its area,

N is the axial force (positive in compression),

b is the width of compressed zone,

f_y and f_c are the steel yield stress and the concrete strength, respectively (in our case, design values),

E_s and E_c are the steel and the concrete modulus of elasticity (design values),

d_b is the diameter of the tension and compression reinforcement,

d_w is the diameter of the stirrups reinforcement,

h is the section height and

c the cover of reinforcement.

For the bottom section of the pier, the yield curvatures are computed as in Table 4.1. The critical value is finally due to the yielding of tension reinforcement.

Table 4.1: Yield curvature at the base of the pier according to Annex 7A of KANEPE

Πίνακας 4.1: Καμπυλότητα διαρροής στη βάση του μεσοβάθρου σύμφωνα με το Παράρτημα 7A του ΚΑΝΕΠΕ

		$(1/r)_y$ (m^{-1})	**ξ_y** (m)
Section yield due to tension reinforcement yielding	**N_{max}**	**0.0027**	**0.444**
	N_{min}	**0.0026**	**0.412**
Section yield due to concrete deformations	**N_{max}**	0.0062	0.357
	N_{min}	0.0066	0.333

4.2. Calculation of yield moment of the section (Annex 7A of KANEPE)

Given the yield curvature, the yield moment $M_{Rd,y}$ is computed as follows:

$$\frac{M_{Rd,y}}{b \cdot d^3} = \left(1/r\right)_y \cdot \left\{ E_c \cdot \frac{\xi_y^{\,2}}{2} \cdot \left(0.5 \cdot (1+\delta') - \frac{\xi_y}{3}\right) + \left[(1-\xi_y)\cdot\rho + (\xi_y - \delta')\cdot\rho' + \frac{\rho_v}{6}\cdot(1-\delta')\right]\cdot(1-\delta')\cdot\frac{E_s}{2} \right\}$$

Thus for N_{max}:

$(1/r)_y$ = 0.0027 m^{-1} the resulting yield moment is: **$M_{Rd,y}$ = 27944 kNm**

The initial effective elastic stiffness of the pier is then calculated as:

$(EI)_{eff}$ = M_y / $(1/r)_y$ = 27944 / 0.0027 = 10349630 kNm^2 ,

while its nominal stiffness is: $(EI)_{nom}$ = 30000 x 2.25 = 67500000 kNm^2

Therefore: $a_{cr} = (EI)_{eff} / (EI)_{nom} = 0.153$

The full curve of moment – curvature of the bottom section of the pier for N_{max}, according to KANEPE, is illustrated in Figure 4.1.

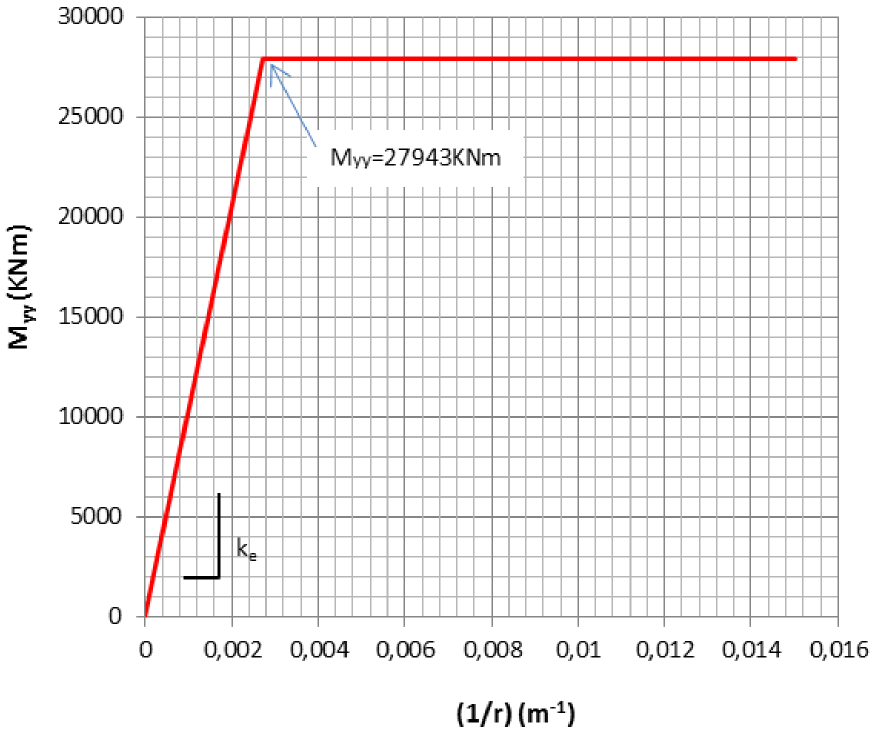


Figure 4.1: Moment- curvature curve at the base of the pier according to KANEPE (for N_{max})

Σχήμα 4.1: Διάγραμμα ροπών-καμπυλοτήτων στη βάση του μεσοβάθρου σύμφωνα με τον ΚΑΝΕΠΕ (για N_{max})

Similarly, for N_{min}:

$(1/r)_y$ = 0.0026 m^{-1} the resulting yield moment is: **$M_{Rd,y}$ = 24728 kNm**

The initial effective elastic stiffness of the pier is then calculated as:

$(EI)_{eff}$ = M_y / $(1/r)_y$ = 24728 / 0.0026 = 9510769 kNm^2 ,

while its nominal stiffness is: $(EI)_{nom}$ = 30000 x 2.25 = 67500000 kNm^2

Therefore: $a_{cr} = (EI)_{eff} / (EI)_{nom} = 0.141$

The full curve of moment – curvature of the bottom section of the pier for N_{min}, according to KANEPE, is illustrated in Figure 4.2.

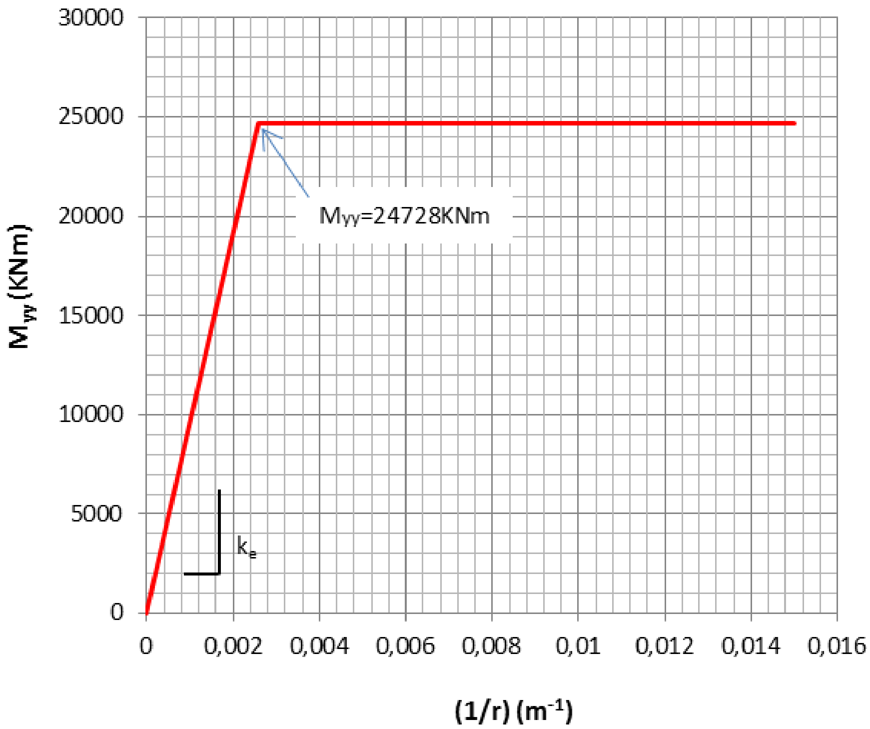


Figure 4.2: Moment- curvature curve at the base of the pier according to KANEPE (for Nmin)

Σχήμα 4.2: Διάγραμμα ροπών-καμπυλοτήτων στη βάση του μεσοβάθρου σύμφωνα με τον ΚΑΝΕΠΕ (για Nmin)

4.3. Linear elastic analysis of the bridge

A finite element model of the bridge is prepared using the software SAP (see Figure 4.3).

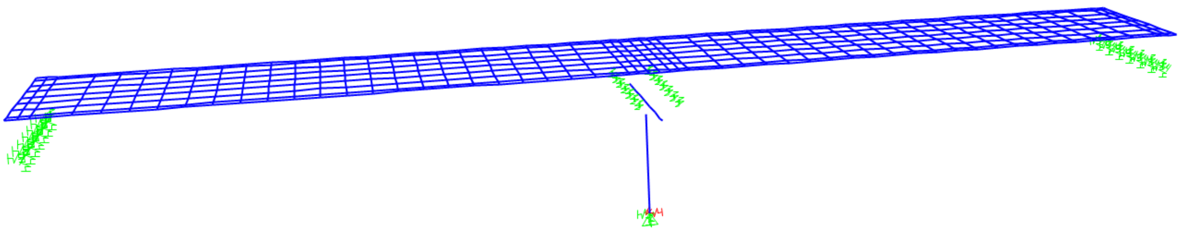


Figure 4.3: Finite element model of the bridge by means of the software SAP

Σχήμα 4.3: Προσομοίωμα πεπερασμένων στοιχείων της γέφυρας με χρήση του λογισμικού SAP

A linear elastic analysis of the bridge is then performed, for a random value of imposed displacements $\Delta = \rho + \varphi y + 0.3\varphi x$, namely for a settlement ρ_{act} and the corresponding rotations φ_y and φ_x , as follows:

ρ_{act}=20cm,

φ_y=0.05x20=1,0degree=0.0175rad, and

φ_x=0.3φ_y=0.005rad

Two load combinations are examined:

- 1.15G + 1.15Δ "+" P "+" 1.35Q
- G + 1.15Δ "+" P

The bending moment M_{act} at the bottom of the pier is defined to be equal to 27132kNm for both loading cases, because, due to the symmetry of the structure, the vertical loads do not produce any bending moments at this section (see Figure 4.3).

The deformed shape of the bridge due to load combination "G+1.15Δ+P" is presented in Figure 4.4.

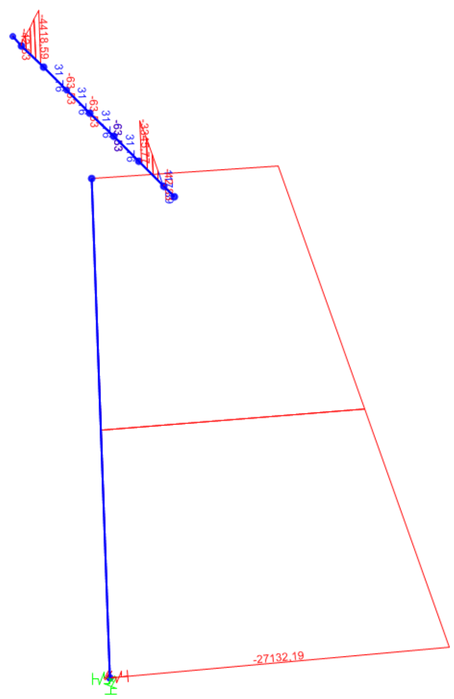


Figure 4.3: Bending moments of the pier of the bridge due to load combination "G+1.15Δ+P"

Σχήμα 4.3: Καμπτικές ροπές στο βάθρο της γέφυρας λόγω του συνδυασμού φορτίσεων "G+1.15Δ+P"

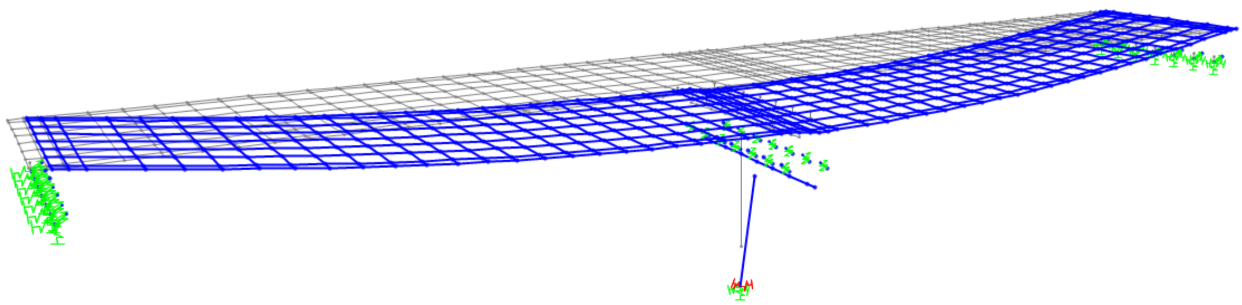


Figure 4.4: Deformed shape of the bridge due to load combination "G+1.15Δ+P"

Σχήμα 4.4: Παραμορφωμένος φορέας λόγω του συνδυασμού φορτίσεων "G+1.15Δ+P"

The max allowable value of settlement ρ_{all} is then calculated as:

For N_{max}: $\rho_{all} = \rho_{act} \times (M_{Rd,y} / M_{act}) = 0.20 \times (27943/27132) = 0.206m$ and

for N_{min}: $\rho_{all} = \rho_{act} \times (M_{Rd,y} / M_{act}) = 0.20 \times (24728/27132) = 0.182m$

The inclination angle corresponding to the max allowable settlement ρ_{all} (for N_{min}) is calculated as:

$\theta_{all} = \theta_{act} \times (M_{Rd,y} / M_{act}) = 0.0119 \times (24728/27132) = 0.0109rad$

The results of the simplified approach are summarized in Table 4.2.

Table 4.2: Allowable values of critical parameters for the design of the innovative bridge derived from the simplified approach (for N_{min})

Πίνακας 4.2: Μέγιστες επιτρεπτές τιμές κρίσιμων παραμέτρων για το σχεδιασμό της πρωτότυπης γέφυρας που προέκυψαν από τον απλοποιημένο σχεδιασμό (για N_{min})

Combination	ρ_{all} (cm)	$\varphi_{y,y}$ (rad)	$\varphi_{x,y}$ (rad)	θ_y (rad)	$M_{Rd,y}$ **(kNm)**
G+1.15Δ+P (N_{min})	**18.2**	**0.016**	**0.005**	**0.0109**	**24728**

It is seen that the results of this simplified method are more conservative than those derived by the more refined method described in chapter 3.

5. Performance criteria of the bridge according to the Codes

According to chapter 2, where an extensive literature survey as well as the provisions of various Codes are presented, limiting values of various types of displacements are generally not directly associated with certain limit states of the structure. Thus, only simplified approaches are possible. Most researchers (see Table 2.1) have the opinion that vertical displacements less than 5cm are tolerable or acceptable. The same value is also suggested by EN1997-1 in Annex H (see Table 2.2). Therefore, this could be the Serviceability Limit State. Furthermore, in Table 2.1, vertical displacements from 5cm up to 10cm are considered harmful but tolerable. This could correspond to an Ultimate State condition.

Assuming that the settlement of the abutment is practically zero, the limiting values of the differential settlements correspond to the allowable vertical displacement of the pier. For concrete simply supported bridges, as in our case, Moulton et al. (1985) set a limit in the allowable angular distortion equal to 0.005 (see Table 2.2), corresponding to a differential settlement of 21cm considering the 42m span of the bridge. Moreover, Moulton et al. (1985) specify a differential settlement of less than 3.81cm to be acceptable for a bridge pier of a concrete bridge following completion, which is a rather conservative value and should not be taken into consideration. In addition, AASHTO sets a limit of 0.008 in the allowable angular distortion of simply supported bridges (see Table 2.2), which corresponds to 33.6cm of differential settlement, a rather large value that should not be taken into consideration, either. Finally, according to EN1997-1 (see Table 2.2), a limiting value of 0.002 is set for the allowable angular distortion of normal structures for serviceability limit state corresponding to 8.4cm of differential settlement. Further, for the ultimate limit state, a limiting value of 1/150 is proposed, corresponding to 28.0cm of differential settlement.

Conclusively, according to the Codes maximum allowable settlements for the serviceability limit state should not exceed 5 to 8cm, while for the ultimate limit state this limit would be in the order of 20cm.

Moreover, according to the Japanese method JBDPA 90-91 (Rossetto et al., 2010) an inclination angle of the pier in the order of 0.01 is proposed to obtain a small damage level (see Table 2.4).

Concerning limiting horizontal displacements, most researchers agree that maximum allowable values should not exceed 2.5cm for the serviceability limit state and 5cm for the ultimate limit state (see Tables 2.1 and 2.2).

6. Conclusion

In this report, the maximum allowable settlement ρ_{all} of the pier of the bridge under investigation, which is a statically determinate, two-span (2×42.00m) concrete bridge, is calculated. The maximum allowable settlement ρ_{all} of the pier is set equal to the settlement which produces moment at the base of the pier equal to the yielding moment M_y. In this way, after liquefaction, damage of the pier will be avoided. Other, secondary structural members that can easily be repaired or replaced (e.g. bearings, joints etc.) are allowed to suffer more serious damage, so they were not taken into account in the calculation of ρ_{all}.

For the above-mentioned analyses, a preliminary design against static and seismic actions should be carried out, in order to initially define the geometrical characteristics and the required reinforcement of the various structural members. In the analyses presented herein, the results of the conventional design were used (see Deliverable D6b).

The fact that no damage of the piers is allowed is completely compatible with the conventional design of this type of bridges against seismic actions, where a practically elastic behavior of the piers is required ($q \leq 1.5$). Nevertheless, it should be noted that the capacity of the pier to carry static and future seismic loads will be reduced, as a result of the additional stress developed, mainly due to the imposed rotations at its base.

For the statically determinate bridge under consideration, it was found that the most critical combination of imposed displacements Δ and corresponding rotations at the footing φ_x and φ, is the one that imposes the maximum rotation φ_y about the weak axis of the pier's section.

The results show that liquefaction-produced settlements at the foundation of the pier in the order of 23 cm can be tolerable, producing only minor damage to the pier, which practically continues to behave elastically. This settlement seems to be compatible with the limits proposed in the Codes for the ultimate limit state, which are in the order of 20 cm.

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