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NONLINEAR BEHAVIOR OF STEEL STRUCTURES

RESEARCH ACTIVITIES AND APPLICATIONS IN ENGINEERING PRACTICE

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Introduction
Limit state design

In recent years **Limit State Design (LSD)** has been adopted by design codes, replacing **Allowable Stress Design (ASD).**

According to limit state design, structures shall be checked in the following two fundamental limit states:

- in the **ultimate limit state (ULS)** and
- in the **serviceability limit state (SLS)**

Introduction
Limit state design

In each limit state a set of performance criteria shall be satisfied regarding the response of the structure to external loads.

- **In order to satisfy the criteria in the ultimate limit state, the structure shall not collapse under "extreme" loads.**
- **In order to satisfy the criteria in the serviceability limit state, the structure shall remain operational for its designated use under daily "ordinary" loads.**

Introduction
Objective of presentation

A numerical methodology is presented for the estimation of the limit load and ultimate strength of steel structures, that are necessary for checking whether criteria in the ultimate limit state are satisfied.

Collapse may occur due to

- ❑ material nonlinearity
- ❑ geometric nonlinearity
- ❑ combination of both of the above two types of nonlinearity

Therefore, numerical analysis for the estimation of the ultimate strength shall be nonlinear, taking into account both types of nonlinearity.

Introduction
Linear elastic material behavior

Hooke's law: $\sigma = E \cdot \varepsilon$

Introduction
Material nonlinearity

Introduction
Geometrically linear behavior

Introduction
Geometric nonlinearity

Introduction
Basic concepts of nonlinear behavior

Almost all structures behave:

- **Linearly** for low load levels
- **Nonlinearly** above a certain load level

Introduction
Basic concepts of nonlinear behavior

Slender structures are flexible and exhibit large deformations, thus:

- **They are more susceptible to geometric nonlinearity, therefore collapse initiation is governed by geometric nonlinearity**
- **As collapse progresses, stiffness decreases further very rapidly, even larger deformations and strains occur, and therefore material nonlinearity develops as well.**

Introduction
Basic concepts of nonlinear behavior

"Stocky" structures (structures of small slenderness) are :

- More susceptible to **material nonlinearity**, therefore collapse initiation is governed by material nonlinearity
- As collapse progresses, stiffness decreases very rapidly, large deformations and strains occur, and therefore **geometric nonlinearity** develops as well.

Introduction
Basic concepts of nonlinear behavior

Influence of initial imperfections

Introduction
Basic concepts of nonlinear behavior

Therefore, for the reliable estimation of a structure's ultimate strength the following should be taken into account:

- **Geometric nonlinearity**
- **Material nonlinearity**
- **Imperfections**

This type of analysis is known as:

Geometry and Material Nonlinear Imperfection Analysis

GMNIA

Introduction
Nonlinearity in modern codes

According to current structural design codes :

- Calculation of **action effects** (internal forces/moments) is achieved by means of linear elastic analysis of the structure subjected to design loads.
- Action effects of each member are compared with corresponding **resistances** (strength), commonly obtained for typical situations from formulas, accounting indirectly for both two types of eventual nonlinearity.

Member strength is commonly calculated by multiplying **section strength**, assuming full utilization of material capacity, with appropriate **reduction factors**, expressing the influence of geometric nonlinearity and imperfections.

Introduction
Nonlinearity in modern codes

Strength of member in compression according to Eurocode 3:

$$N_{b.Rd} = \frac{\chi \cdot \beta \cdot A \cdot f_y}{\gamma_{M1}}$$

section strength

material nonlinearity

χ: reduction factor accounting for global buckling

β: reduction factor accounting for local buckling

Geometric non linearity + imperfections

Introduction
Nonlinearity in modern codes

Advantages of methodology adopted by the codes:

- A strongly nonlinear phenomenon is approximated by means of a simple linear analysis
- Readily available commercial software can be used
- Design checks are relatively simple
- No special know-how is needed
- Reliable results are obtained for **common** structural systems and for members with **conventional** cross sections

Therefore, application of this methodology is recommended whenever this is possible.

Introduction
Nonlinearity in modern codes

Alternatively, most current codes allow the use of nonlinear numerical analysis for the estimation of ultimate strength.

When is such an analysis necessary?

- **For research purposes**

or

- **For design in special cases, for example:**
  - **Members with unusual cross sections**
  - **Unconventional structural systems**

In those cases buckling curves and interaction formulas proposed in the codes are not valid,

&

approximations that are often made in order to use them nevertheless, may lead to unreliable results.

Introduction
Nonlinear numerical analysis

- Commercial software for nonlinear analysis with the finite element method (FEM) is easily available.
- However, engineers who are using such software must possess adequate knowledge regarding:
  - Setting up numerical models
  - Selecting proper analysis methods and proper parameters of numerical solution algorithms
  - Understanding / interpreting the results

These issues are addressed in this presentation.

Finite element method
Nonlinear analysis

Principle of virtual work:

$$\int_{{}^0V} {}^{t+\Delta t}_{\;\;0}S_{ij}\,\delta\,{}^{t+\Delta t}_{\;\;0}\varepsilon_{ij}\,d^0V = {}^{t+\Delta t}R$$

${}^{t+\Delta t}_{\;\;0}S_{ij}$ **is the second Piola-Kirchhoff stress tensor**

${}^{t+\Delta t}_{\;\;0}\varepsilon_{ij}$ **is the Green-Lagrange deformations tensor**

Equilibrium equation in matrix form:

$$\underbrace{\left({}^t_0K_0 + {}^t_0K_u + {}^t_0K_g\right)}_{{}^t_0K}\Delta U = {}^{t+\Delta t}R - {}^t_0F$$

${}^t_0K$ ⇨ **tangential stiffness matrix**

ΔU ⇨ **increment of displacement vector**

Finite element method
Linear buckling analysis

At buckling the tangential stiffness becomes zero, and therefore the condition for instability is

$$\det\left({}_0^tK\right) = \det\left({}_0^tK_0 + {}_0^tK_u + {}_0^tK_g\right) = 0$$

which leads to the following linear buckling formulations

$$\left({}_0^tK_0 + {}_0^tK_u + {}^t\lambda\,{}_0^tK_g\right)\varphi_i = 0$$

$$\left({}_0^tK_0 + {}^t\lambda\left({}_0^tK_u + {}_0^tK_g\right)\right)\varphi_i = 0$$

$^t\lambda$ ⇨ **load parameter**

φ_i ⇨ **ith mode**

Neglecting the initial displacements matrix ${}_0^tK_u$**, the classical buckling formulation is obtained as:**

$$\left({}_0^tK_0 + {}^t\lambda\,{}_0^tK_g\right)f_i = 0$$

Proposed numerical methodology
Model setup

Step 1. Setup of appropriate numerical model

i. Basic model checks (e.g. reactions, eigenmodes, periods) by means of linear, static and modal analysis.

ii. Confirmation of finite element mesh through convergence checks (e.g. successively decreasing element dimensions to one half).

iii. Validation of results through comparison with experimental results (preferably) and/or available analytical and/or numerical solutions.

iv. Prediction of all possible failure modes (e.g. local or lateral torsional buckling cannot be predicted by a beam element model).

Proposed numerical methodology
Linear buckling analysis

Step 2. Performance of linear buckling analysis (LBA)

i. Quick and low computational cost type of analysis.

ii. Critical loads that are computed from LBA are an initial indication and in most cases an upper limit of actual strength.

iii. In order to trigger all possible failure modes, buckling modes that are obtained from LBA are used as imperfections for the subsequent nonlinear analyses.

Proposed numerical methodology
Nonlinear analysis

Step 3. Nonlinear analysis

Types of nonlinear analysis:

- **Without imperfections:**
  - **Material nonlinear analysis (MNA)**
  - **Geometric nonlinear analysis (GNA)**
  - **Geometric & material nonlinear analysis (GMNA)**
- **With imperfections:**
  - **Material nonlinear analysis with imperfections (MNIA)**
  - **Geometric nonlinear analysis with imperfections (GNIA)**
  - **Geometric & material nonlinear analysis with imperfections (GMNIA)**

Proposed numerical methodology

Nonlinear analysis

- Initial types of analysis can be used in order to investigate separately the influence of each type of nonlinearity and of imperfections on the response of the structure.
- Finally GMNIA is recommended for the evaluation of ultimate strength.

Proposed numerical methodology
Imperfections

- **Shape of Imperfections:**

  In case no measured data of actual imperfections are available, a linear combination of buckling modes should be used.

- **Magnitude of imperfections:**

  Important only in cases of steep unstable post-buckling equilibrium path (e.g. thin shells).

  Relevant codes provide recommended magnitude of imperfections for specific cases.

Proposed numerical methodology
Nonlinear analysis algorithms

Most common method ⇨ Newton-Raphson Method (NR)

Proposed numerical methodology
Nonlinear analysis algorithms

Modified Newton-Raphson method (MNR)

Proposed numerical methodology
Nonlinear analysis algorithms

Equilibrium path beyond limit point ⇨ arc-length type methods

Crisfield

Riks-Wempner

Proposed numerical methodology
Interpretation of results

Equilibrium path, in combination with images of stresses and deformations at characteristic points along the path, in order to understand the physical phenomena that occur during progressive increase of loads.

Application examples
Software

- ADINA
- ABAQUS
- SAP 2000 NL

Application examples
Software validation

Pinned column

Imperfection:

- 1st mode shape
- Magnitude according to EC3

Application examples
Software validation

Simple cable net under dynamic harmonic loads P=sin(ωt)

$\Delta t_1 \approx T/50 = 0.01$sec

$\Delta t_2 \approx T/100 = 0.005$sec

Application examples
from research activity

- **Wind turbines towers**
  - Optimization of stiffening of manholes
- **Built-up steel columns**
  - Formulation of design rules for arbitrary boundary conditions and for combined axial and bending actions

Wind turbine towers
Description of problem

From the doctoral thesis of Christoforos Dimopoulos

Influence of manholes

- ❑ **Stress concentrations**
- ❑ **Local buckling**

Wind turbine towers
Reinforcement of manholes

- Objective of stiffening
  - To control local stresses
  - To provide adequate lateral support to the shell thus establishing adequate resistance against local buckling
- Research objective
  - Evaluation of the efficiency of different stiffening schemes of manholes at wind turbines towers
- Methodology
  - Experimental study and parallel numerical simulation for calibration
  - Parametric numerical study
  - Design guidelines

Wind turbine towers
Experimental work

Geometry of specimens

Wind turbine towers
Numerical model

- Three types of nonlinearity where taken into account
  - Geometric non-linearity (necessary in order to predict local buckling)
  - Material nonlinearity
  - Contact (bolts and flanges)
- Initial geometric imperfections where found to have small effect
- Residual stresses where ignored

Wind turbine towers
Numerical model

Simplified numerical model for the estimation of support (testing frame) flexibility

Detailed numerical model for the estimation of load – displacement curves and strains

Zoom A

Zoom B

56 bolts and 4 flanges under contact

Wind turbine towers
Manhole reinforcement

Stiffeners considered as flanges under compression and classified as category 1 cross-sections according to EC3 provisions

Area of manhole A_0

Area of frame stiffener A

Wind turbine towers

Comparison of experimental & numerical results

No manhole | **Unreinforced manhole** | **Reinforced manhole**

Wind turbine towers
Influence of manhole and stiffening

Experimental

Numerical

Wind turbine towers
Numerical analysis results

A

C

B

Wind turbine towers
Experimental / numerical results

Deformation after collapse

No manhole

Reinforced manhole

Unreinforced manhole

Wind turbine towers
Alternative stiffening schemes

Frame stiffener

Stringer stiffener

Frame & stringer stiffeners

Frame & stringer stiffeners with extra stiffeners in between

Wind turbine towers
Alternative stiffening schemes

Negligible influence of additional stiffeners

Wind turbine towers

Comparison of alternative stiffening schemes

Required A/A_0 ratio for different stiffening types

Wind turbine towers
Dimensioning of alternative stiffening schemes

Stiffening type	Quality A	Quality B	Quality C
	A/A_0=0.6 (b/t= 2.979,b=175mm) A/A_0=0.6 (b/t= 11.916,b=350mm)	A/A_0=0.6 (b/t= 2.979,b=175mm) A/A_0=0.8 (b/t= 8.937,b=350mm)	A/A_0=0.8 (b/t= 2.234,b=175mm) A/A_0=0.8 (b/t= 8.937,b=350mm)
	A/A_0=1.2 (b/t=1.49,b=175mm) A/A_0=0.8 (b/t=8.937,b=350mm)	A/A_0=1.2 (b/t=1.49,b=175mm) A/A_0=1.0 (b/t=7.15,b=350mm)	A/A_0=1.4 (b/t=1.277,b=175mm) A/A_0=1.2 (b/t=5.958,b=350mm)

Required A/A_0 ratio for stringer and frame stiffeners

Built-up columns
Problem description

Recommended in cases of:

- ❑ **Large height / long span**
- ❑ **Heavy loads**

From the doctoral thesis of Kostas Kalocheretis

Research objective:

- ❑ **Arbitrary boundary conditions**
- ❑ **Axial + bending actions**
- ❑ **Design guidelines**

Built-up columns
Interesting issues

- Influence of shear deformation
- Interaction of global buckling, local buckling and plasticity

Built-up columns
Laced – Numerical modeling

- Shell finite elements for flanges
- Beam finite elements for lacing

Finite element model

Global buckling mode

Local buckling mode

Built-up columns
Laced – Numerical modeling

Equilibrium path

Deformations and von Mises stresses (kN/m²) at characteristic points A and B

Built-up columns
Battened – Numerical modeling

- Shell finite elements for both flanges and battens
- Detailed modeling of welded connections

Finite element model

Global buckling mode

Local buckling mode

Built-up columns
Battened – Numerical modeling

Equilibrium path

Deformations and von Mises stresses (kN/m²) at characteristic points A and B

Application examples
from engineering practice

- **Protection hangar for Aristotle's Lyceum**
  - Design of pylon with varying cross section
  - Design of main arch with varying cross section

- **Thessaloniki-Skopje & Komotini – Alexandropoulis – Kipi buried pipelines for fuel transport**
  - Recommendations of protective measures in areas of potential landslides and seismic fault activation

Aristotle's Lyceum protection hangar
Architectural proposal

Architectural design team

K. Karadimas

D. Loukopoulos

K. Vrettou

Ch. Papadimitriou

L. Stavropoulou

Aristotle's Lyceum protection hangar
Description of structural system

Steel roof consists of 6 parallel arch shaped main frames and is suspended by cables from 6 pylons.

Pylon ⇨ **circular hollow section, varying over the height**

Arch-shaped main frames ⇨ **welded I section strengthened with sideplates near the support**

Aristotle's Lyceum protection hangar

Pylon design

Circular hollow section, varying over height

All cross section are classified to class 1 or 2 in order to avoid local buckling

Height of pylon

h=25m

Pylon sections

- **max**

  D=800mm/t=14mm

  (middle section)

- **min**

  D=300mm/t=30mm

  (end sections)

x (mm)	D (mm)	τειμλ (mm)
0	300.0	30
500	339.2	
1000	376.8	
1500	412.8	25
2000	447.2	
2500	480.0	18
3000	511.2	
3500	540.8	16
4000	568.8	
4500	595.2	16
5000	620.0	
5500	643.2	16
6000	664.8	
6500	684.8	14
7000	703.2	
7500	720.0	14
8000	735.2	
8500	748.8	14
9000	760.8	
9500	771.2	14
10000	780.0	
10500	787.2	14
11000	792.8	
11500	796.8	14
12000	799.2	
12500	800.0	

Aristotle's Lyceum protection hangar

Pylon design

Modeling ⇨ beam elements

shell elements

- **No local buckling among the first ten modes.**
- **First (flexural) buckling mode as imperfection pattern.**

First critical buckling mode

Aristotle's Lyceum protection hangar
Pylon design

Aristotle's Lyceum protection hangar

Main frame design

Cross sections from pylon to column:

- **Double RHS700/210/15/30**
- **RHS700/455/15/30**
- **RHS700/400/15/30**
- **I630/400/15/35**

Cross sections of class 1 or 2 were used in order to avoid local buckling

Aristotle's Lyceum protection hangar
Main frame design

Modeling

- Two main frames with shell elements
- Transverse and braced connection members with beam elements
- Retaining cables with nonlinear, tension only, prestressed truss elements

- No local buckling among the first ten modes.
- The first global buckling mode (lateral-torsional) as imperfection pattern.

Aristotle's Lyceum protection hangar

Main frame design

Equilibrium paths from different types of analysis

Aristotle's Lyceum protection hangar

Main frame design

Point A-deformation

Point A-stresses

Point B-deformation

Point B-stresses

Deformation and stress distribution at characteristic points (1/3)

Aristotle's Lyceum protection hangar

Main frame design

Point C-deformation

Point C-stresses

Point D-deformation

Point D-stresses

Deformation and stress distribution at characteristic points (2/3)

Aristotle's Lyceum protection hangar
Main frame design

Point D-stresses (detail)

Deformation and stress distribution at characteristic points (3/3)

Buried pipelines for fuel transport
Problem description

High Pressure Natural Gas Pipeline

Komotini – Alexandropoulis – Kipi

Buried pipelines for fuel transit

Problem description

Oil pipeline

Thessaloniki-Skopje

Buried pipelines for fuel transit
Problem description

Scope:

To evaluate the consequences of potential landslides or fault activation, triggered by an earthquake, and to propose protection measures.

Buried pipelines for fuel transit

Pipeline modeling

2-node isoparametric 3-D beam elements, outside the critical region, where gradual reduction of stresses and strains is expected

4-node isoparametric shell elements in critical region, for more accurate estimation of stress peaks

Rigid element at interace between shell and beam elements

Buried pipelines for fuel transit

Soil modeling

Buried pipelines for fuel transit

Soil modeling

Axial springs

Transverse horizontal springs

Transverse vertical springs

Buried pipelines for fuel transit

Numerical results

Buried pipelines for fuel transit
Numerical results

Stresses of axial soil springs along the pipeline

Longitudinal normal strain along the pipeline

Buried pipelines for fuel transit
Proposed measures

- Excavation expansion
- Fill in with soft material

Buried pipelines for fuel transit

Proposed measures

- Wrapping the pipeline with a friction reducing geotextile

Final remarks

- Nonlinear numerical modeling with the finite element method is a powerful calculation tool for the structural engineer…
- … but a very **dangerous** one.
- Its correct use demands a very good knowledge of the physical problem and of the theoretical background of the numerical method.
- Its use is recommended for research purposes or for unconventional design problems, for which simpler methods are not adequate.
- Validation of results via comparison with experimental results or well known analytical or numerical solutions is highly recommended.