Alma Mater Studiorum – Università di Bologna

DOTTORATO DI RICERCA IN

INGEGNERIA STRUTTURALE ED IDRAULICA

Ciclo XXIV

Settore Concorsuale di afferenza: 08/B2 – SCIENZA DELLE COSTRUZIONI

Settore Scientifico disciplinare: ICAR/08

NONDESTRUCTIVE EVALUATION OF CONCRETE COMPRESSION STRENGTH BY MEANS OF ARTIFICIAL NEURAL NETWORK (ANN)

Presentata da: MARIO BONAGURA

Coordinatore Dottorato

CHIAR.MO PROF. ING. ERASMO VIOLA

Relatore

PROF. ING. LUCIO NOBILE

Esame finale anno 2012

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Abstract

The evaluation of structural performance of existing concrete buildings, built according to standards and materials quite different to those available today, requires procedures and methods able to cover lack of data about mechanical material properties and reinforcement detailing. This issue is more relevant when seismic zones are concerned and structural strengthening needs to prevent failures due to earthquakes. Recent seismic codes give relevance to procedure and methods to establish the performance levels of existing structures. To this end detailed inspections and test on materials are required. It is recognised that different levels of knowledge(KL) can be reached depending on available technical design reports, structural drawings and material acceptance data. As a consequence tests on drilled cores are required; on the other end, it is stated that non-destructive testing (NDT) cannot be used as the only mean to get structural information, but can be used in conjunction with destructive testing (DT) by a representative correlation between DT and NDT.

The aim of this study is to verify the accuracy of some formulas of correlation available in literature between measured parameters, i.e. rebound index, ultrasonic pulse velocity and compressive strength (SonReb Method). To this end a relevant number of DT tests and NDT tests has been performed on many school buildings located in Cesena (Italy). The above relationships have been assessed on site correlating ND test results to strength of core drilled in adjacent locations.

Nevertheless, concrete compressive strength assessed by means of NDT methods and evaluated with correlation formulas has the advantage of being able to be implemented and used for future applications in a much more simple way than other methods, even if its accuracy is strictly limited to the analysis of concretes having the same characteristics as those used for their calibration. The above limitation warranted a search for a different evaluation method for the non-destructive parameters obtained on site. To this aim, the methodology of neural identification of compressive strength is presented. Artificial Neural Network (ANN) suitable for the specific analysis were chosen taking into account the development presented in the literature in this field. The networks were trained and tested in order to detect a more reliable strength identification methodology.

Chapter 1

Italian Standards and guidelines

Sommario

Il patrimonio edilizio in cemento armato, realizzato in gran parte tra gli anni ‘60 e ‘70, rappresenta ad oggi oltre il 50% del patrimonio ad uso abitativo. Il raggiungimento dei 40 anni di servizio per una struttura in cemento armato rappresenta il superamento di una soglia al di sopra della quale si rendono necessari controlli. In questo discorso generale sul patrimonio edilizio nazionale, non si deve dimenticare che larga parte del territorio è a rischio sismico (l’OPCM n°3274 del 2003 ha portato il numero il numero dei comuni classificati come sismici dal 37% al 58% dei comuni italiani) e solo una parte degli edifici in tali aree è stato progettato utilizzando criteri antisismici. Le Norme Tecniche per le Costruzioni (NTC 2008) suddividono il patrimonio edilizio italiano in due grandi raggruppamenti: edifici nuovi ed edifici esistenti. Per la valutazione degli edifici esistenti la normativa italiana consente di assumere, nelle verifiche di sicurezza, un adeguato valore del fattore di confidenza in base al livello di conoscenza acquisito della struttura analizzata. Per la scelta del tipo di analisi e dei valori dei fattori di confidenza vengono definiti i tre livelli di conoscenza seguenti (NTC, 2008): LC1 Conoscenza Limitata; LC2 Conoscenza Adeguata; LC3: Conoscenza Accurata.

Gli aspetti che definiscono i livelli di conoscenza sono: la geometria; i dettagli strutturali e i materiali.

1.1 Introduction

The safety of existing buildings is a fundamental task in Italy if we consider on the one hand the seismic vulnerability of our territory and on the other hand the high value of our historical, architectural and artistic heritage.
A recent research carried on by Censis (Centro Studi Investimenti Sociali) has showed that the reinforced concrete buildings, built mostly between the 60s and 70s, represents nowadays more than the 50% of the Italian housing heritage. The 40 years service for a reinforced concrete structure represents the overcoming of a threshold that makes necessary specified controls.
In this situation, we have not to forget that all the Italian territory is classified as seismic zone (OPCM n. 3274 of 2003 has changed the number of cities classified as seismic from 37% to 58%) and only a part of the buildings has been designed using seismic criteria.

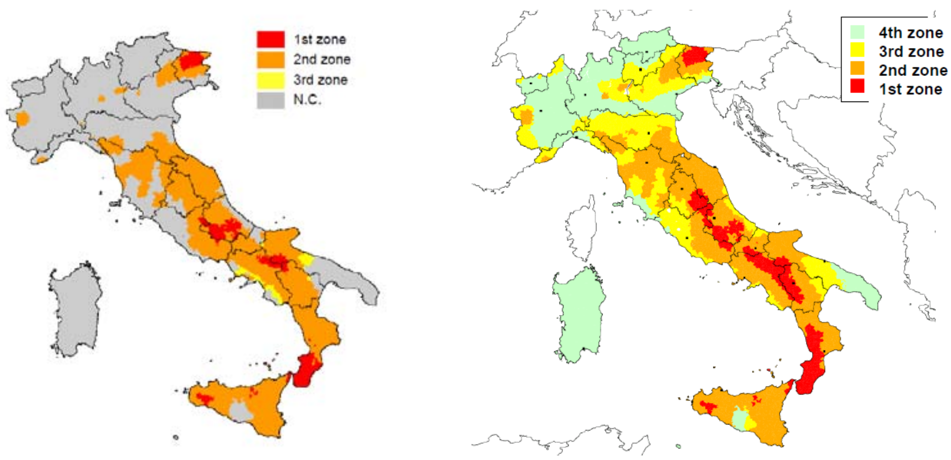

Fig. 1.1: Comparison between seismic zoning: on the left side the first seismic zoning based on seismic hazard studies (until 1984) and on the right side the new one (introduced by OPCM n. 3274 in 2003).

The Technical Standards for Construction (NTC 2008) subdivided the Italian building heritage into two groups: new buildings (projected in accordance with seismic criteria) and existing buildings. It is defined existing building the one that has, to the date of drafting the safety evaluation or project, the structural frame fully realized.
The logic process of evaluation and seismic adaption can be summarized into 5 main phases:

1. Knowledge of the structure (i.e. geometry, mechanical characteristics of structural materials and their conditions);
2. Definition of performance requirements (i.e. seismic risk of the site, intended use and level of seismic protection required/accepted);
3. Evaluation of the existing structure (i.e. definition of the model study, seismic analysis and safety verification);
4. Project of seismic structure improvement (in relation to existing constraints and performance requirements);
5. Assessment of the appropriate structure.

1.2 Evolution of the Italian technical standards for constructions

In order to better understand the character of the new Italian technical standards (NTC 2008), we should make a brief state of the art of the evolution time of the national technical standards for construction. Unfortunately the main character role in this evolution is covered by the elevated number or earthquakes over the years that have imposed many reflections on this field.

Before the unification of Italy, following the earthquake of 1783 in Calabria, (which caused about 30,000 casualties), the government of the "Kingdom of the Two Sicilies" adopted a regulation which made reference to the reconstruction rules for selecting sites on which to rebuild, the types of structural and construction details. In 1859, after the earthquake of Norcia, the Papal States issued strict rules to norm the phase of reconstruction (they set the characteristics of the foundation, maximum height of buildings and building materials). Following the earthquake of Messina and Reggio Calabria in 1908 (evaluated as the most catastrophic earthquake occurred since the unification of Italy with the death of almost 90,000 people), *Royal Decree April 18, 1909 n. 193* was issued.

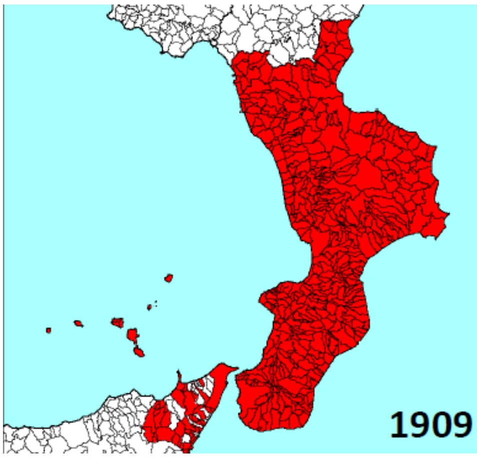

Fig. 1.2: The first seismic zoning in Italy (Calabria Region)

The latter one listed a few hundred of cities in Sicily and Calabria in which was placed the obligation to respect the technical standards introduced by the Royal Decree (R.D.) for the construction or repair of buildings. Since this R.D., it was inaugurated the seismic classification of the territory and was started the production of norms that would follow the further development of science and structural engineering.

Over the next 30 years, the Italian

standards were focused again on seismic hazard and few other aspects. The *Ministerial Decree of October, 30, 1912* approved the rules for testing and acceptance of timber materials for buildings constructed by the Ministry of Public Works. The *Royal Decree of July, 25, 1913 n. 998*, dealt with the hygienic standards in the construction of major public works. The *Legislative Decree* (L.D.) *n. 1526 of 1916*, has quantified the seismic forces and their distribution along the height of the building. The Royal Decree of December, 31, 1923 n. 3046 was focused on "*standardization of materials that can be used by the administrations of the Italian State*" while the R.D. of two years later (*August, 7, 1925 n. 1616*) was issued to regulate testing of electrical equipment in constructions. The successive *Royal Decree n.431 of 1927* has classified the territory as seismic, introducing two levels of seismicity: high (1st zone) and moderate (2nd zone) on the basis of the observed damages and the subsequent seismic forces in each of them. *The R.D. n. 640 of 1935* represented, in many ways, a great step forward, with the enactment of specific technical guidelines and the requirement for municipalities to adopt their own building codes. Furthermore, the evaluation of the overall behavior of frame structures was required for the first time according to the theory of hyperstatic elastic systems.

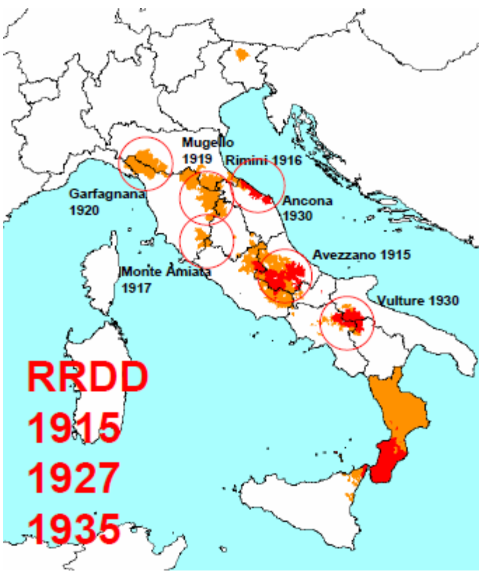

Fig.1.3: The relationship between Royal Decrees and seismic events.

Since 1962, with the approval of the *Act n. 1684* has provided that the seismic standards should be adhered to in the "cities subject to intense seismic activity" and not just in "cities affected by the earthquake". Despite all, nothing had changed from the previous practice of classify only as a result of earthquakes. The next two laws passed during the 1970s represent the first standards nationally imposed. The *Act n.1086 of November 5, 1971* regulates the reinforced concrete, prestressed and steel structures. The *Act n. 64 of 1974*, represents a milestone in the way of evolution of the actual legislation. In fact, it has allowed an easier successive upgrade of the standards, giving to special Ministerial Decrees (M.D.) the task to regulate, in terms of security, the different sectors of construction. This innovative decision providing an easier upgrade later in the standards. For this reason, in the following years, technical standards for buildings, bridges, dams and foundations were produced without having to issue new law. It was also introduced, for the first time, a particularly incisive system of verifications and repression of violations. Between the end of 1970s and the first years of 1980s[1], a need to rationalize the seismic classification of Italian territory was felt. In fact, it was not homogeneous in its distribution due to its nature evolved by successive aggregations of cities affected by earthquakes. The *Decree of the Ministry of Public Works of July 14, 1984* represents the latest in a series of decrees in the first years of 1980s which have been reshaped the limits of seismic classification that are still partially in force.

During the next 20 years, a series of decrees who tries to adjust all the more important aspects of building were approved. The *Ministerial Decree of December, 12, 1985* approved the technical standards for piping. The *Ministerial Decree of November, 20, 1987* regulates the planning, execution and testing phases of masonry buildings and their consolidation while the *Ministerial Decree of December, 3, 1987* regulates the prefabricated buildings. The following year, the *Ministerial Decree of March, 11, 1988* analyzes geological and geotechnical aspects. The *Ministerial Decree of May 4, 1990* regulates the planning, execution and testing phases of road bridges. This leads to the *Ministerial Decree of January, 16, 1996* that

[1] After the catastrophic Irpinia Earthquake in 1980 (3500 deaths), the Italian Government issued a research group to continuously update the knowledge about seismic hazard: the National Group Against Earthquakes, GNDT (by the Italian National Research Council, CNR).

represent the latest version of the technical standard on the calculation, construction and testing of concrete, prestressed and steel structures. The latter decree also indicates the general criteria for verifying safety of structures, loads and overloads.

Italian technical standards for constructions were deeply innovate in March 2003 with the approval of the *Order of the President of the Council of Ministers (OPCM) n. 3274* [2]. The aims of this ordinance was to collect and summarize all the rules concerning different aspects of building design, and adjust the general and technical construction criteria for the design, implementation and testing phase of ordinary and special buildings (such as bridges, dams, aqueducts, etc..), as well as to classify the entire national territory according to the seismic hazard. The OPCM. n. 3274 also radically changes the theoretical basis on which the previous legislation was based on construction. The main point of reference becomes the *Eurocodes*, for which still retains many differences. The basic concept introduced by the Ordinance[3] is the *Capacity Design System*. This method asserts that the structures designed according to the new standards have margins of resistance that enable them to resist without collapse to seismic actions higher that of the project ones. Subsequently, the Parliament has promulgated the *Decree-Law of May 28, 2004, n. 136,* in which it has been settled seismic and hydraulic technical standards for construction, as well as the establishment of technical standards for the design, construction and adjustment of dams, bridges and the work of foundations. In the same decree was agreed that the priorities for actions must be ensured for school and sanitary buildings. Therefore, in order to implement the Decree-Law n. 136, was approved the *Ministerial Decree of September 14, 2005* that issued

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[2] Immediately after the earthquake of October, 31, 2002 which hits the territories between Molise and Puglia, the Civil Protection adopted this ordinance, in order to provide an immediate response to the need to update the seismic zoning and earthquake standards.

[3] The ordinance n. 3274 has been subjected to numerous and significant corrections, made mostly with the OPCM n. 3316 of October, 2, 2003, the OPCM n. 3379 of November, 5, 2004 and the OPCM n. 3467 of October, 13, 2005.

the technical standards for construction. The aim of this M.D. was to bring together into a single technical text related to the design and construction and at the same time achieving the standardization and rationalization. This Ministerial Decree represented a complete development of the complex legislation of constructions, in relation to both structural design and the main civil engineering works and materials characteristics. It represented also a substantial update of the national legislative framework, previously based on the fundamental laws n. 1086/71 and n. 64/74.

Due to both complexity and inapplicability of some parts and continuous updating of coefficients and benchmark parameters of the last decree, the *Ministerial Decree of January 14, 2008* was approved. Nowadays this decree, together with the applicative circular approved in April 2008, represent the latest version of the technical standards for constructions (NTC). The new NTC issued in 2008 introduce several significant changes about the responsibilities assigned to various actors in the field of concrete construction. The most important and innovative changes[4] regard mostly the obligations of the designer that, in addition to the calculation of structural elements, shall provide guidance on the installation, the seasoning and the characteristics of the concrete. In particular, he have to indicate the value of R_{ck} that takes account of the durability of the structure in relation to the environment where the work is realized and the service life specified in the project. Along with these strict obligations, the Italian technical standards for construction leave to the designer the responsibility to decide whether to refer also to other technical literature or international standards (i.e. UNI EN).

[4] Other important changes regards the design of buildings in seismic zones that should ensure the achievement of Earthquake Performance Levels in the occurrence of earthquakes with different intensity (Earthquake Design Levels) with the “Performance Based Design”.

1.3 Plan of investigation of reinforced concrete structures

The planning of investigation of a reinforced concrete building with reference to the standards previously mentioned, it should perform with following phases:

1. Definition of the survey program;
2. Historical analysis and collection of the original project;
3. Execution of on-site inspections.

1.3.1 Definition of the survey program

The draft program of investigation consists of a complete historical (see 1.3.1) and structural (see 1.3.2) description of the building, how to perform the in-site and laboratory tests. It must also contains all the data obtained by the tests and the document tables and plants that indicate the structural elements investigated.

1.3.2 Historical analysis and collection of the original project

The historical analysis and collection of the original project drawings is a fundamental phase to obtain a correct recognition of the existing structural system and its state of preservation. This phase must be conducted with all relevant Local Authorities in order to obtain the following documentations[5]:

- Structural projects;
- Architectural drawings;

[5] With particular reference to the guidelines issued by Toscana Region in the program VSCA (Seismic Vulnerability of reinforced Concrete buildings) and the OPCM n. 3274 of March 20, 2003.

- Calculation reports;
- Construction documents (i.e. books of measures and test certificates);
- Historical information regarding the project (i.e. current standards at the time of the execution of the buildings);
- Historical information regarding the different phases of construction (i.e. identification of construction company, different variations during construction, etc..);
- Substantial and no substantial changes (i.e. functional distribution, proposed use, etc..);
- Maintenance conditions.

These source materials will allow the identification of the main aspects of construction as:

- Identification of the structural frame and verification of compliance with the regularity criteria specified in regulations;
- Identification of the foundation structures;
- Identification of the soil categories;
- Information on the geometrical dimensions of the structural elements, the amount of reinforcement rods, the connections between different elements and the mechanical characteristics of materials employed;
- Information on possible local defects of materials;
- Information about possible defects in the structural details;

- Description of both the current and future intended use for the building;
- Information regarding the type and extent on any damages and repairs previously made;
- Any mechanical tests already carried out both in-situ and in laboratory to characterize the property of building or materials.

1.3.3 Execution of on-site inspections

The execution of on-site inspections represent a very important step to characterize the existing buildings and should be aimed to determine the following aspects:

a) Verification of the correspondence between the actual state of the building and the structural design (in the case of they can be found), otherwise execution of a new structural relief with:

    - verification of geometry and construction details;
    - verification of the size of the structural elements;
    - verification by means of covermeter (and/or removal of the concrete cover) of the amount and placement of the main reinforcement rods and stirrups, their closure and densification in proximity of nodes[6].

b) On-site taking samples for the typological characterization of the floors and exterior panels, aimed to determining the weights to compute in the loads analysis;

[6] This practice constitutes a preliminary essential phase to be performed in order to avoid cutting portions of reinforced rods during the taking of samples.

c) Analysis of the degradation state of structures (i.e. for reinforced concrete: visual inspection of its quality, any removal of the concrete cover by rusty rods, etc..);

d) Detection of crack patterns;

e) Detection of any damage phenomena caused by plant installations;

f) Preliminary identification of the structural elements able to be investigated (i.e. emerging beams, columns with free opposite sides).

1.4 Evaluation of existing buildings with NTC 2008

The problems proposed by the existing buildings mainly concern the evolution of standards during the time and in particular in the evaluation of mechanical characteristics of structural materials. With regards to the latter one, the Italian standard links the evaluation of the mechanical characteristics of materials determined by in situ tests to three different coefficients: the confidence factors. In fact, NTC 2008 allows to assume an appropriate value of the confidence factors based on the level of knowledge (KL) of the structure analyzed. To choose the type of analysis and the values of the confidence factors, the following three levels of knowledge are defined: KL1 Limited Knowledge, KL2 Normal Knowledge, KL3 Full Knowledge.

1.4.1 Levels of knowledge (KL)

On the basis of the analysis carried on in cognitive phases, it will be identified the *levels of knowledge* (KL) of the different parameters employed in the model (geometry, structural details and materials), and defined the related *factors of confidence* (FC) ,to be used as further partial coefficients of safety that take into account the gaps in knowledge of model parameters. The aspects that define the levels of knowledge are: geometry (i.e. the geometric characteristics of the structural elements), structural details and materials (i.e. mechanical properties of materials).

Level of knowledge	*Geometry*	*Structural details*	*Materials characteristics*	*Methods of analysis*	*FC*
KL1	*Original drawings with visual survey in-site or complete relief.*	*Simulated projected and limited in-situ testing.*	*Usual values for the construction practice of the time and limited in-situ testing.*	*Linear analysis Static or Dynamic.*	*1,35*
KL2		*Structural details incomplete with limited or extended in-situ testing.*	*Values obtained from the original drawings or certificates with limited or extensive in-situ testing.*	*All*	*1,20*
KL3		*Complete construction drawings with limited or exhaustive in-situ testing.*	*Values obtained from the original test certificates with extensive or exhaustive in-situ testing.*	*All*	*1,00*

Table 1.1: Levels of knowledge depending on the information available and consequent methods of analysis allowed.

If you wish to reach a *Limited Knowledge* level (KL1), the three main parameters mentioned above, should be managed as follow[7]:

- *Geometry:* geometry of the structure is known according to a in-situ relief or from the original drawings. In the latter case, a visual survey must be carried out to verify the correct correspondence

[7] Instructions of Technical Standard for Construction (NTC 2008), Italy, 2008

between drawings and building (i.e. number of structural elements, main dimensions of columns and beams, span dimensions etc..). The data collected on the size of the structural elements will be sufficient to enable the development of a model suitable for a linear analysis.

- *Structural details:* details are not available from the constructional drawings and must be determined on the basis of a simulated project carried out according to the practice of the period of construction. Limited in-situ tests are required for the reinforcement rods and for the most important connection elements. The data collected will be sufficient to enable verification of local strength or the development of a model suitable for a linear analysis.
- *Material characteristics:* if no information is available on the mechanical characteristics of the materials employed, neither by the constructional drawings nor by the test certified, it has to be taken into account of the usual values for the construction practice of the time validated by limited in-situ tests on the most important elements.

If you wish to reach a *Normal Knowledge* level (KL2), the three main parameters mentioned above, should be managed as follow:

- *Geometry:* geometry of the structure is known according to a in-situ relief or from the original drawings. In the latter case, a visual survey must be carried out to verify the correct correspondence between drawings and building. The data collected on the size of the structural elements, together with those concerning the structural details, will be sufficient to enable the development of a model suitable for a linear and non-linear analysis.
- *Structural details:* details are known from an extensive in-situ testing program or partially available from the incomplete original construction drawings. In the latter case, a limited in-situ survey is

required to verify the reinforcement rods and the most important connection elements. The data collected will be sufficient to enable verification of local strength or the development of a model suitable for a non-linear analysis.

- *Material characteristics:* information concerning mechanical characteristics of materials are available, based on construction drawings, original test certificates or extensive in-situ testing program. In the first case it has to be carried out also limited in-situ testing program. If the data obtained from the in-situ tests are smaller than those available from the construction drawings or by the original certificates, it has to be carried out extensive in-situ testing program. The data collected will be sufficient to enable verification of local strength or the development of a model suitable for a non-linear analysis.

If you wish to reach a *Full Knowledge* level (KL3), the three main parameters mentioned above, should be managed as follow:

- *Geometry:* geometry of the structure is known according to a in-situ relief or from the original drawings. In the latter case, a visual survey must be carried out to verify the correct correspondence between drawings and building. The data collected on the size of the structural elements, together with those concerning the structural details, will be sufficient to enable the development of a model suitable for a linear and non-linear analysis.
- *Structural details:* details are known from an exhaustive in-situ testing program or available from the original construction drawings. In the latter case, a limited in-situ survey is required to verify the reinforcement rods and the most important connection elements. The data collected will be sufficient to enable verification of local

strength or the development of a model suitable for a non-linear analysis.

- *Material characteristics:* information concerning mechanical characteristics of materials are available, based on construction drawings, original test certificates or exhaustive in-situ testing program. In the first case it has to be carried out also extensive in-situ testing program. If the data obtained from the in-situ tests are smaller than those available from the construction drawings or by the original certificates, it has to be carried out exhaustive in-situ testing program. The data collected will be sufficient to enable verification of local strength or the development of a model suitable for a non-linear analysis.

1.4.2 Geometry (carpentry)

The geometry of the building investigated (i.e. determination of the geometric characteristics of its structural elements) must be determined by analyzing the following project documents:

- *Original carpentry drawings:* they show the geometry of the structure, the structural elements and their dimensions. They also allow to identify the structural frame that resists to the horizontal and vertical actions.

- *Construction or executive drawings:* they describe the geometry of the structure, the structural elements and their dimensions. They also allow to identify the structural frame that resists to the horizontal and vertical actions. In addition, they hold information about the amount, the placement and the construction details of all the reinforcement rods, as well as the nominal mechanical characteristics of the materials used.

- *Visual survey:* it is used to verify the effective correspondence between the actual geometry the available original carpentry drawings. It includes the sample survey of the geometric characteristics of some elements. In the case of undocumented modifies that occurred during or after the construction period, it will carried out a complete relief as described in the following section.

- *Complete relief:* it is used to produce a complete documentation (carpentry drawings) if the original ones are missing or if is found any incongruity between carpentry drawings and actual geometry of the structure. The material produced has to describe the geometry of the structure, the structural elements and their dimensions. It also allows to identify the structural frame that resists to the horizontal and vertical actions with the same degree of detail of the original drawings.

To correctly identify the geometry of the structure, the obtained data must include the following parameters:

- *Identification of the structural frame that opposed to the horizontal actions in both directions;*

- *Framework of the floors;*

- *Geometrical dimensions of beams, columns and walls;*

- *Possible eccentricity between columns and beams in their connections.*

1.4.3 Construction details

The construction details of the building investigated must be determined through the following phases:

- *Simulated project:* it is used in absence of the original construction drawings to define the amount and the placement of the reinforcement rods in all the structural elements. It is also employed to determine the most important characteristics of the principal connections. It is performed according to technical standards and the construction practice in use at the building time.
- *Limited in-situ tests:* They are used to verify the correspondence between the reinforcement rods or the characteristics of the principal connections actually present in the building and those reported in the constructional drawings, or obtained by the simulated project.
- *Extensive in-situ tests:* They are used as an alternative to the simulated project (in accordance to limited in-situ tests) when the original construction drawings are not available or incomplete.
- *Exhaustive in-situ tests:* They are used to reach a full knowledge level (KL3) when the original construction drawings are not available.

The in-situ testing programs are carried out on an appropriate amount of structural elements for each different type (beams, columns, walls, etc..) as shown in the table 1.2.
To correctly identify the construction details, the obtained data must include the following parameters:

- *Amount of longitudinal reinforcement rods in beams, columns and walls;*
- *Number of elements and structural detail of transverse reinforcement rods in the critical zones and beam-column nodes;*

- *Amount of longitudinal reinforcement rods in the floors that contributes to the negative moment in T-beams;*
- *Support lengths and constraint conditions of the horizontal elements;*
- *Thickness of concrete cover;*
- *Length of the overlapping zones of reinforcement rods.*

Tests and Relief	*Relief (on construction details)*	*Test (on materials)*
	For each "primary" element type (beams, columns, etc..)	
LIMITED	The amount and placement of reinforcement rods is verified for at least 15% of the elements.	1 specimen of concrete every 300 m^2 of concrete mixture used for floor.
EXTENSIVE	The amount and placement of reinforcement rods is verified for at least 35% of the elements.	2 specimens of concrete every 300 m^2 of concrete mixture used for floor.
EXHAUSTIVE	The amount and placement of reinforcement rods is verified for at least 50% of the elements.	3 specimens of concrete every 300 m^2 of concrete mixture used for floor.
N.B.	It is allowed to replace some destructive tests (no more than 50%) with an higher number of non-destructive tests (at least three times the number of DT substituted) calibrated on destructive ones.	

Table 1.2: Definition of the different levels of relief and testing programs for reinforced concrete structures.

1.4.4 Material characteristics

The mechanical characteristics of the materials used in the building investigated must be determined through the following phases (the appropriate testing program for different cases is shown in Table 1.2):

- *Concrete:* the evaluation of the mechanical characteristics is obtained coring samples subsequently tested in compression up to failure.
- *Steel:* the evaluation of the mechanical characteristics is obtained taking a set of reinforcement rod samples subsequently subjected to tensile tests up to failure. The data obtained must allow the determination of the yield strength and the ultimate strain strength (with the exception of the case in which those values are available by test certificates consistent with those required for a new constructions).
- *Connections of steel elements:* the evaluation of the mechanical characteristics is obtained taking a set of samples subsequently subjected to tensile tests up to failure. The data obtained must allow the determination of the yield strength and the ultimate strain strength.
- *Non-destructive test methods:* NDT methods are allowed only for attested reliability. They cannot be used on their own or as a replacement of the above mentioned. They can also be used to integrate the destructive tests (DT), provided the NDT results are calibrated to those obtained with DT methods. For the concrete, it is important to use testing methods able to limit the influence of carbonation phenomena on the evaluation of the compressive strength of concrete.
- *Limited in-situ tests:* they allow to integrate the information on material properties obtained by the standards in force at the time of the construction, or by the nominal characteristics reported on construction drawings, or by the original test certificates.

- *Extensive in-situ tests:* they allow to obtain information on material properties in absence of both construction drawings and original test certificates. They are also used when the data obtained from limited in-situ tests are lower than those reported in construction drawings or original test certificates.
- *Exhaustive in-situ tests:* they allow to obtain information on material properties in absence of both construction drawings and original test certificates. They are also used when in required a level of Full Knowledge (KL3) and the data obtained from limited in-situ tests are lower than those reported in the construction drawings or original test certificates.

1.4.5 Factors of confidence (FC)

The *factors of confidence (FC)* are determined to define the material strengths to be used in further verifications or in the analytical models in absence of more exhaustive evaluations. The mean strengths obtained by both in-situ tests and additional information are divided for the factors of confidence. They can also be different for each type of materials (on the basis of statistical considerations conducted on a set of meaningful data for the considered elements).

1.5 NDT methods in Italian standards for construction

The Non-Destructive Testing (NDT) are the complex of in-situ examinations, testing and inspections, carried out on the whole building or parts of it, that do not damage the material or cause any local damage usually accepted. The results obtained from these test type represent the basis to evaluate the quality of a particular material, to examine the integrity or to diagnose and research the causes of deterioration. The development of these NDT methods in construction is due to two main factors: firstly, the increased interest in condition monitoring of existing buildings, and secondly the increased focus of the recent technical standards on the characteristic of "*durability*" to ensure the buildings. These test type were initially designed for testing machinery, engines and heir components. Then, given the results achieved and the technological development in this field, the Italian standard has begun to employ NDT as an useful tool to evaluate the seismic vulnerability of buildings[8]. Only in recent years, they are acquiring their own fundamental importance. The main problem with regard to NDT has always been about the reliability of the results and their evaluation. These two factors represent the basis of structural integrity assessment of a component and/or materials, and, consequently, help to establish the level of quality of each product.

In this context we need a technical standards able to unifying both results and methods employed. This role is played by the international unification institute, such as UNI (*Italian organization for Unification*), ISO (*International Organization for Standardization*) and CEN (*Comité Européen de Normalisation*). The role of this unification institute mainly concerned with the regulation of aspects of calibration, execution and interpretation of results obtained through these tests.

The evolution of Italian standards that deals with NDT methods regards mainly the last twenty years, and can be summarized as follows:

Ministerial Decree 09/01/96

- Section 3.1: ("*Static testing - General requirements*"): part of the activities and obligations imposed on the tester, in addition to those considered "*mandatory*", is left to the "*discretion*" of the tester "*to conduct these investigations that help to form the belief of the security work*" among them, even the "*non-destructive testing on structures*";

OPCM n.3274/2003

- Point 11.2.3.3 ("*Levels of knowledge*"): in order to adequately support the safety assessment of existing buildings, is provided the use of "*in-*

[8] See also DM 09/01/1996 and OPCM n. 3274/2003.

situ tests" and "*non-destructive test methods*" provided that are of "*documented reliability* ." It states that "*the same cannot be used in complete replacement of the above*" (i.e. compression tests), "*but only recommended to their integration, provided the results are calibrated to those obtained with destructive tests*."

Ministerial Decree 14/09/05

- Point 8: ("*Static testing - General requirements*"): replace quite similar the previous indications presented in Section 3.1 of the D.M. 09/01/96. The main difference between this two Ministerial Decrees mainly concerned the purpose with the introduction of the concept of "durability" of the building (this purpose wasn't indicated in the previous ones).

- Point 11.1.6 ("*Verification of the actual concrete strength*") the use of NDT is provided in cases where other tests do not give satisfactory results or when "*arise doubts about the quality and responsiveness of the concrete values of resistance*" or when it "*becomes necessary to evaluate retrospectively the actual properties of concrete*". Also in this case, it is specified that "*these tests must not, in any case, intended to substitute the acceptance tests*."

Similar elements to the previous standard can also be found both in the actual Italian technical standards for construction (NTC 2008) and in the *Eurocodes* and other European standards of technical character.

Chapter 2

Non-destructive testing for the evaluation of concrete

Sommario

Su un campione di edifici scolastici realizzati tutti tra gli anni 60 e 70 è stata condotta una campagna di prove sperimentali costituita da prove distruttive e non distruttive preliminare alla valutazione della vulnerabilità sismica. In prima istanza sono state effettuate accurate indagini pacometriche volte alla determinazione delle aree di calcestruzzo non interessate dalle prove di caratterizzazione meccanica. L'indagine pacometrica è la metodologia non distruttiva per la localizzazione delle armature negli elementi in calcestruzzo armato. Questo metodo si avvale del principio della misurazione dell'assorbimento del campo magnetico, prodotto dalla stessa apparecchiatura, che viene evidenziato tramite sistema analogico o digitale. Successivamente si è provveduto ad eseguire prove di tipo non distruttivo per la valutazione della resistenza caratteristica a compressione con il metodo combinato *SonReb* che deriva dall'utilizzo di due strumenti: lo Sclerometro e il Rilevatore Ultrasonoro. L'indagine sclerometrica consente di determinare la durezza superficiale del calcestruzzo mediante la misura del rimbalzo di un'asta proiettata sulla superficie della struttura stessa. Attraverso curve di correlazione si può inoltre stimare la resistenza a compressione del calcestruzzo. L'indagine ultrasonica invece consente la valutazione

dell'uniformità di calcestruzzo delineando al contempo le zone di degrado o di scarsa qualità. Questa metodologia di prova non distruttiva viene anch'essa impiegata per stimare la resistenza a compressione del calcestruzzo mediante opportuna correlazione del valore di velocità di propagazione degli ultrasuoni nel materiale. La valutazione della resistenza a compressione del calcestruzzo è solitamente basata sulle relazioni empiriche tra risultati ottenuti mediante prove distruttive e parametri ottenuti attraverso l'impiego dei test non distruttivi precedentemente citati. Per la valutazione non distruttiva della resistenza del calcestruzzo esistono numerose formulazioni in letteratura corrispondenti alle curve di isoresistenza, dalle quali dati i valori di velocità media di propagazione dell'onda ultrasonora e l'indice medio di rimbalzo si ottiene il valore di resistenza del calcestruzzo: il candidato si è concentrato sull'applicazione delle formule di *R. Giacchetti*, *L. Lacquaniti* (1980); *J. Gasparik* (1992) e *A. Di Leo*, *G. Pascale* (1994) come consigliato dalle Indicazioni Tecniche della Regione Toscana.

2.1 Introduction

The increasing technological development in the construction field offers, nowadays, the possibility of analyze buildings with non-invasive experimental techniques able to provide as much information as possible. The conceptual basis of these techniques is certainly found in the "*experimental method*[1]". The benchmark of this method is the determinism that is the concatenation required between a phenomenon and the causes that produce it and must be divided into three main phases:

[1] The experimental method is the scientific method introduced by F. Bacon and G. Galileo in the seventeenth century. This method is based mainly on observation of physical phenomena, the use of mathematics and the reproducibility of the experiment. Through the observation of phenomena and the testing repeated the scientist can interpret and determine the mathematical relationships underlying natural phenomena. The scientist formulates scientific hypotheses and subject them to the control of the experimental method. The confirmed hypothesis by repeated experiments turn into scientific laws.

- *Testing program* (i.e. the definition of what we need to test);
- *Execution of tests;*
- *Analysis of results* (i.e. the verification of the expected forecasts in relation with any differences between forecasts and results obtained).

Thus the use of NDT methods leads to the estimation of the mechanical characteristics of the material from the measurement of a non-destructive parameters appropriately correlated, as generally between them there is no direct physical correlation. Therefore, as an indirect methods, it is important to calibrate the results on an adequate number of destructive tests. In addition from a careful analysis of the literature it can be seen as often these parameters are influenced by specific factors linked to the state of conservation of materials (i.e. in concrete structures the phenomenon of carbonation alters the values of surface hardness). Thereof it can be easily deduced that in order to obtain a reliable result is not sufficient to use only one technique, but it is advisable to integrate multiple techniques able to complement each other.

Thus the candidate has led a campaign of experimental tests composed of non-destructive and destructive tests prior to the evaluation of seismic vulnerability on a series of reinforced concrete school buildings built between the 60s and 70s. Firstly it has been carried out a detailed covermeter investigations due to determine the areas of concrete not affected by the mechanical characterization tests. The covermeter survey is a non-destructive methodology employed to localize the steel reinforcement in concrete elements. This method consists of applying an magnetic field to the element under inspection and measuring the absorption of this field by means of the use of a scanning magnetic field sensor.

Then the candidate proceeded to perform non-destructive tests to evaluate the characteristic concrete compressive strength with the combined method SonReb resulting from the use of *Rebound hammer* and the *Ultrasonic Pulse Velocity* (UPV) detector. The Rebound hammer test allows to determine the surface hardness of concrete by measuring the rebound of a metal rod projected on the surface of the investigated material. Through correlation curves it can be also estimated the compressive strength of the concrete since the mean value of Rebound Index (RI).

Instead UPV test allows the evaluation of the uniformity of concrete outlining at the same time decayed areas or surface of poor quality. Also this NDT method is used by itself to estimate the compressive strength of concrete by means of appropriate correlation of the value of ultrasound propagation velocity in the material. In literature there are many formulations able to ensure this correlation: among the available the candidate has chosen the formulas proposed by *Hisham, Y. Qasrawi* (2000); *R. Giannini* (2003) and *M. Bilgehan*, *P. Turgut* (2010).

The evaluation of the concrete compressive strength by means of the combined method *SonReb* is usually based on empirical relationships between the results obtained with destructive tests (i.e. samples taken from the structure and subjected to compression tests) and parameters obtained through the use of NDT methods previously cited (i.e. RI and UPV). There are many formulations in literature corresponding to the correlation curves, to which it is possible to evaluate the value of concrete strength since average data of UPV and RI: the candidate has focused his attention on the application of formulas of *R. Giacchetti*, *L. Lacquaniti* (1980); *J. Gasparik* (1992) and *A. Di Leo*, *G. Pascale* (1994) as recommended by the Technical Guidelines of the Tuscany Region.

2.2 Covermeter survey

2.2.1 Generality

Safety structural evaluation of reinforced concrete buildings includes both the estimation of in-situ mechanical characteristics of concrete and the relief of steel reinforcement (i.e. longitudinal bars and stirrups) in structural elements (i.e. beams, columns, slabs, retaining walls, etc...). In the diagnosis of existing buildings, if the position and dimensions of the concrete elements are easily measurable, it cannot be said the same as regards both the arrangement and the diameter of the reinforcing bars contained in the elements themselves.
It is a fundamental task to recovery a large number of information about the steel reinforcements in a concrete structure in order to verify its actual condition. In fact it is well known that an adequate cover to the steel reinforcement is necessary to ensure that the steel is maintained at a sufficient depth into the concrete, away from the effects of carbonation or aggressive chemicals. On the other hand, an excessively deep cover can favors both the increasing of crack widths and the decreasing of lever arm.
For the non-destructive determination of this information, it should be adopted an appropriate techniques such as, for example, the *covermeter*[2] *survey*. The presence of reinforcement elements in concrete can be detected by the influence that reinforcing steel has upon a magnetic field induced by the covermeter. The covermeter operates by measurement of the reluctance of the magnetic circuit.

Fig. 2.1. A covermeter survey

[2] In 1951 the "covermeter" was developed in England by the Cement and Concrete Association in conjunction with the Cast Stone and Cast Concrete Products Industry.

2.2.2 Normative references

The following referenced documents are indispensable for the correct application of this type of investigation. The first to provide a detailed standard on how to carry out a correct covermeter survey were the British Standards (BS) in 1988, later integrated into European standards (UNI EN). Since the first versions of the standards emphasizes the importance of preventive calibration phase[3] of the instrument in order to optimize the results of the survey itself. The most important Italian, European and International standards that governing the test execution, instrument calibration and interpretation of results are:

- *BS 1881-204:1988* Testing concrete. Recommendations on the use of electromagnetic cover meters;
- ACI 228.2R: 1998 (Revisited 2004) Nondestructive Test Methods for Evaluation of Concrete in Structures
- *DIN 1045:3-2008* Concrete, reinforced and prestressed concrete structures - Part 3: Execution of structures;
- *UNI EN ISO 15548-1:2009* Non destructive testing – Equipment for eddy current examination – Part. 1: Instrument characteristics and verification;
- *UNI EN 13860-2:2003* Non destructive testing – Eddy current examination – Equipment characteristics and verification – Probe characteristics and verification;
- *UNI EN 13860-3:2004* Non destructive testing – Eddy current examination – Equipment characteristics and verification – System characteristics and verification;
- *UNI EN 1992-1-1:2005* Eurocode 2.

[3] British Standard suggests a basic calibration procedure involving a cube of concrete of given proportions with reinforcing bars at specified distances from the surface.

2.2.3 Scope

This test helps to identify steel reinforcement within the structure and is used to determine steel reinforcement bar spacing, size of reinforcement bars and the depth of reinforcement from surface without damaging the structure under examination. Finding information on the reinforcement rods is an important task also for the execution of quality control and assessment of the mechanical characteristics of concrete using destructive and/or non-destructive test methods. In fact, in this kind of analysis is essential to work in areas where there are no reinforcing bars in order not to influence the results of the investigations themselves. Then, in order to perform properly a non-destructive and/or destructive test methods on reinforced concrete structures, firstly it must be determined the areas where there are steel rods so as to exclude them to:

a) carry out the Rebound Hammer tests (i.e. to avoid any alterations of the rebound index value);

b) carry out UPV tests (i.e. to avoid any alterations of the propagation velocity as a result of the presence of material with different transmission properties compared to concrete);

c) carry out drilling core samples (i.e. to avoid any alterations in subsequent compression tests).

Fig. 2.2. Different aims for a covermeter survey: a) to estimate the actual arrangement, number and diameter of reinforcing bars; b) to identify concrete areas on which drilling core samples; c) to identify concrete areas on which carrying out NDT methods.

2.2.4 Test equipment and applications

The covermeter, also called *pachometer* (*Fig. 2.3*), is a portable reinforcement bars locaters instrument that can be easily used on-site. It is composed by a control unit able to emit/read the electromagnetic field and an emitter/receiver magnetic field probe (characterized by a layer of paramagnetic material). Its use allows a reliable reconstruction of reinforcement, together with the concrete cover to the bar and/or the size of the bar.

Fig. 2.3. A covermeter survey equipment used to locate reinforcement.

The position and direction of steel reinforcement can be determined by moving the search probe of over the surface of the investigated element until the control unit shows a maximum deflection. At this position the reinforcing bar is below and parallel to the length of the search head. It is very important to remember that the probes are highly directional. This means that all probes have both a long and a short axis. A distinct maximum in induced current is observed only when the long axis of the probe and the reinforcement are aligned and when

Fig. 2.4. A covermeter survey on the surface to be investigated.

the probe is directly above the reinforcement. Marking with chalk the position of the reinforcing bars as determined by the instrument it is possible to establish also the spacing of reinforcing bars embedded in concrete. It can be also used to search other metal objects embedded inside the structure (i.e. pipes, electrical wires, rods, etc...) or – in a more general way – for the location of curbs, lintels, columns and slabs of ribs that are not directly visible. The traditional type of covermeter is based on the principle of *magnetic induction* that is, on the perturbation of a magnetic field generated by a probe that is made to slide in a continuous way on the surface to be investigated (*Fig. 2.4*). The search probe is able to work with the perturbation because it is made up with materials that have a different and higher magnetic permeability than that of the concrete in which are inserted the steel reinforcements. The probe that slides on the investigated surface is a coil traversed by an alternating current of constant frequency which generates an alternating magnetic field (*Foucault induced currents* or *eddy currents*). The presence of metal objects in the area of influence of the magnetic field alter the potential difference of the coil in accordance with the principle of magnetic induction. The alteration of the voltage – showed by means of a instrument's graduated scale (located in the control unit) – is strongly influenced both by the thickness of the metallic object and the concrete cover. As depth/spacing ratios increase, it becomes more difficult to discern individual bars.

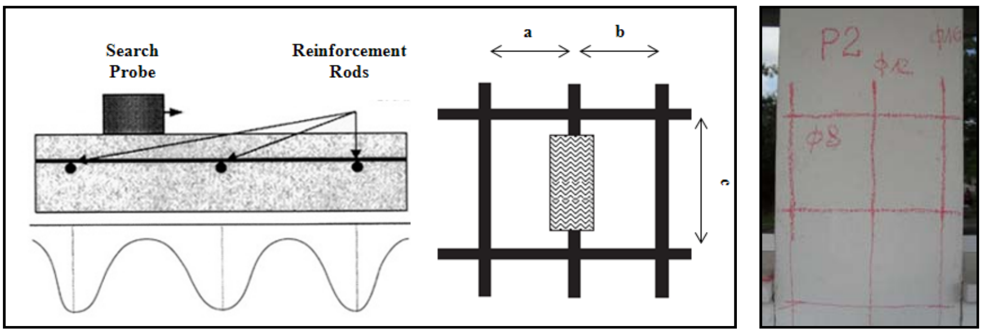
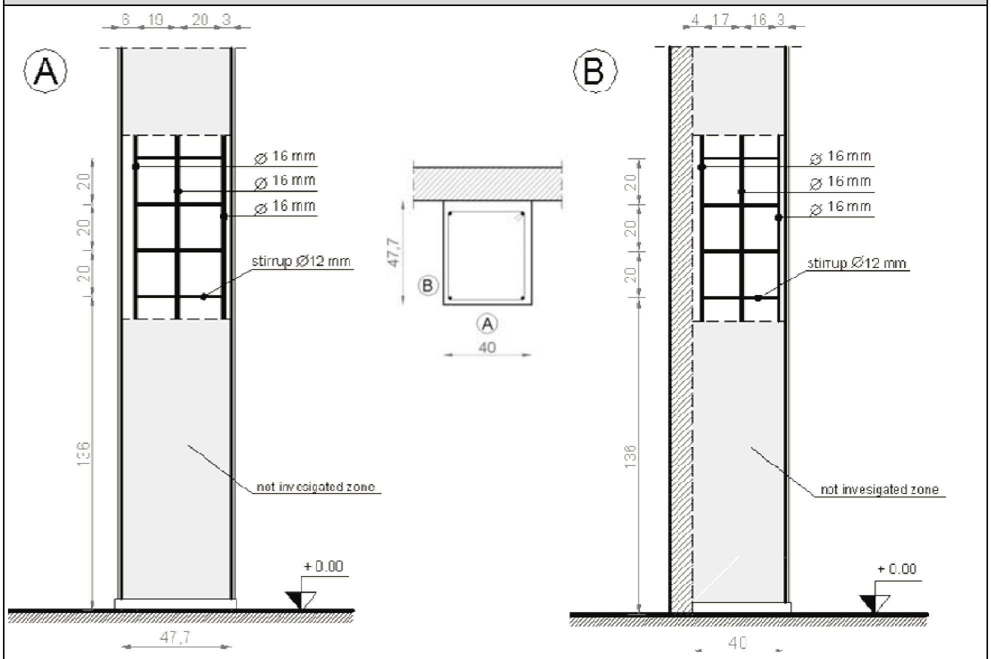


Fig. 2.5. Alteration of the signal due to the slide of the probe on the structural elements: scheme and output of the survey.

For this reason, in the case of high concrete cover or in the presence of strongly armed structures (i.e. prestressed structures) becomes difficult the use of covermeter. The sinusoidal shape of the recorded data (see *Fig. 2.5*) represents the peaks associated with any reinforcing bars and, by measuring the distance of each peak to the edge of the chart paper graph, depth of cover can be determined from an opportunity calibration curve. Some drawbacks of this methods concern the operating temperature range of investigated elements: in fact such instruments will not function satisfactorily at temperatures below freezing.

Position	Description
Point P2 Column Ground Floor	– plasterwork (thickness 1,5 cm); – vertical reinforcement rods Ø 16 mm and Ø 12 mm; – horizontal reinforcement rods Ø 8 mm, steps of 20 cm; – mean concrete cover 3,5 cm.
Scheme	



Note. All measures are in [cm]

2.3 Rebound Hammer test

2.3.1 Generality

The *Rebound hammer* is a hand held instrument used for testing the quality of hardened concrete in an existing structures. The test device was developed in 1948 by Ernst Schmidt [4] at the Swiss Federal Materials Testing and Experimental Institute in Zurich. The instrument is based upon the *rebound principle*. The latter one says that the rebound of an elastic mass impacting on a surface is a function of the hardness of the surface itself. Hence the harder the surface, the higher is the rebound value obtained. The development of this new device draws its inspiration from tests carried out to measure hardness of metals. For this reason, this instrument can be considered as an outgrowth of the *Scleroscope*[5] test, which measures the superficial hardness of a material considering the rebound height of a diamond-tipped hammer, or mass, that is dropped from a fixed height above the test surface.

Due to its simplicity and low cost, the rebound hammer represents the most widely NDT methods used for concrete evaluation. While the test appears simple, there is no direct relationship between the rebound number and the strength of concrete. Since 1958, many authors [6] has attempted to establish a correlation between the Rebound Index (RI) and the concrete surface hardness as measured by the Brinell method with poor results.

Fig. 2.6. Rebound hammer test

[4] E. Schmidt, *Der Beton, Prufhammer*. Schweiz, Bauz. (Zurich), 1958, p. 378

[5] In Greek, the word "σκληροσ" means "hard".

[6] J. Kolek, *An appreciation of the Schmidt rebound hammer*, Mag. Concr. Res. (London), 10(28), 27, 1958.

2.3.2 Normative references

Due to the high degree of uncertainty associated with strength predictions based upon the rebound hammer test, it took many years before the development of the test method and its first certified standards. The first one was issued by British Standards Institution in 1971, followed by the American Society for Testing and Materials in 1975 (ASTM C 805 75-T), and by the International Union of Laboratory and Experts in Construction Materials, System and Structures (RILEM) in 1977. Nowadays many international institute of standardization has adopted standards for the test execution, instrument calibration and interpretation of results, even if this test methods is not recommended by itself. The most important are:

- *BSI BS 1881-202:1986* Methods of Testing concrete. Part 202: Recommendations for Surface Hardness Testing by Rebound Hammer;
- ASTM C805/C805M-08 Standard Test Method for Rebound Number of Hardened;
- *DIN 1048-2:1991* Testing concrete; testing of hardened concrete (specimens taken in situ);
- *UNI EN 12504-2:2001* Testing concrete in structures – Part. 2: Non destructive testing – Determination of rebound number;
- *ACI Committee 228: 2003* In-place methods for determination of strength of concrete, Technical Committee Doc. 228.1R-03, American Concrete Institute;
- *UNI EN 1992-1-1:2005* Eurocode 2;
- *UNI EN 13791: 2007* Assessment of in-situ compressive strength in structures and precast concrete.

2.3.3 Scope

This test type is used to determine the rebound surface hardness of a concrete element by measuring the rebound distance of a mass projected on the surface of the element itself. Since there is some correlation between the surface hardness and mechanical strength, often the rebound hammer is used alone to estimate the values of concrete compressive strength. In fact, the correlation between surface hardness and compression strength depends directly and/or indirectly on many factors (i.e. the hardness of the aggregate elements, finishing and compaction of the cortical layer, curing the surface, etc..) that can influence the effectiveness itself. Usually, for determining the compressive strength with the application of this test method, an estimated accuracy in the order of 15 ÷ 20% is achieved, due to the non-homogeneous and isotropic behavior of material.

Thus, because of the many factors besides concrete strength than can affect rebound number, this NDT methods can be used only for quick surveys in order to assess the uniformity of concrete and areas characterized by the presence of poor quality or deteriorated concrete.

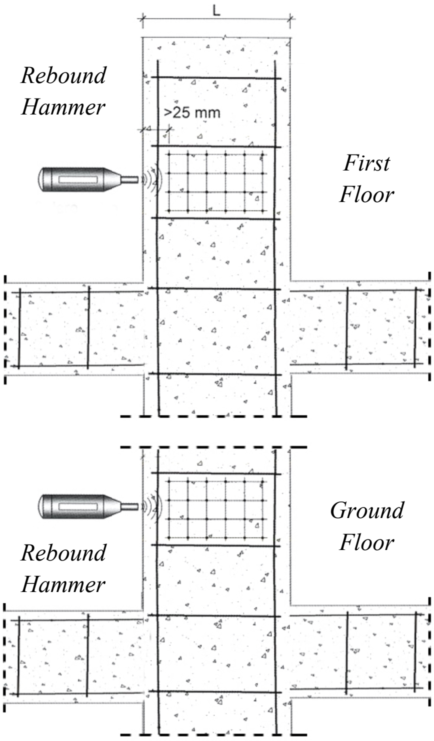


Fig. 2.7. Typical survey carried out with rebound hammer to assess the uniformity of concrete along different floors in the same element.

Fig. 2.8. Surface preparation.

It is also important to remember that there are numerous types of rebound hammers depending on the material to be investigated, mainly characterized by different values of impact energy of the elastic mass on the surface to be investigated (Table 2.1). The one commonly used for concrete investigations is the N-type.

Type	*Impact energy*	*Test (on materials)*
Linear L	0,075 kg · m	Thin concrete elements; Lightweight aggregate concrete or artificial stone and shock sensitive.
M	3 kg · m	Massive concrete (street paving, airport runways, etc. ..).
N	0,225 kg · m	Concrete common-size (buildings, bridges, etc. ..).
P (angular)	0,09 kg · m	Light elements of concrete; Coatings of low class.
NA	0,225 kg · m	Underwater construction;
NR	0,225 kg · m	As the N-type with automatic recording.
LB	0,075 kg · m	Variant of L-type for tiles and handmade clay.
PT	0,09 kg · m	Variant of the P-type for materials with very low resistance.

Tab. 2.1. Different types of Rebound hammer according to the material to be investigated and impact energy of the elastic mass.

2.3.4 Test equipment and applications

The test equipment is composed of a spring-loaded hammer linked to a graduated scale able to reflect the surface hardness of the investigated material. Nowadays there are several models of rebound hammer, all of them characterized by some essential parts: the outer body, the hammer, the plunger, the spring, and the slide indicator (*Fig. 2.9. a*).

To perform the test, the plunger is extended from the body of the instrument, producing a latch mechanism able to grab hold of the hammer (*Fig. 2.9. a*). Then, pushing the instrument toward the concrete surface, which causes the stretching of the spring linked to the hammer and the body (*Fig. 2.9. b*). When the instrument is bring to the limit of the plunger, the latch is released and the hammer is throw toward the surface (*Fig. 2.9. c*). The hammer strikes the plunger and it turns back (*Fig. 2.9. d*). The *rebound index* is expressed on the scale reported on the outer body as a percentage of the initial extension of the spring.

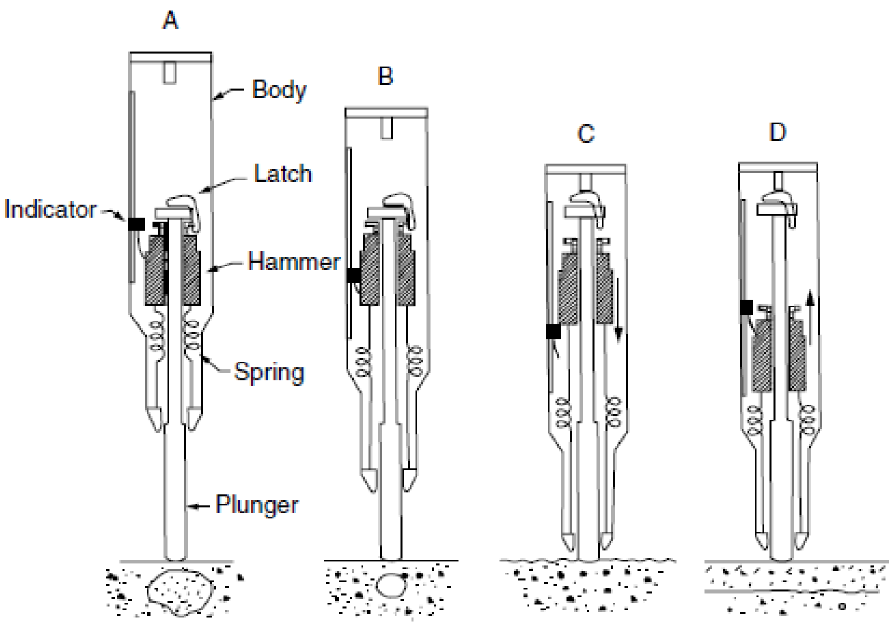


Fig. 2.9. Schematic cross section of N-Type rebound hammer showing operating applications

2.3.5 Principal drawbacks

As mentioned before, there are many factors that affect the rebound number for a given strength of concrete. Most of them are strictly connected to the properties of concrete, other derive mainly from the non correct application of the test equipment. In practice the test is very dependent upon the following aspects:

- Age of the concrete, because of the maturation of the concrete greatly influences the surface hardness;
- The high presence of cement paste determines an underestimation of the resistance due to the greater deformability of the concrete surface while, conversely, low content for the concrete is less deformable due to the resistance offered by the aggregates;
- High water/cement ratio reduces the estimate of the resistance (at the opposite, higher behavior for low water/cement ratio);
- Aggregate type influences the strength of the concrete that depends directly on the type of aggregates used. Therefore it is necessary to carry out specific calibrations when using special aggregates which tend to overestimate the resistance;
- Concrete Moisture can cause reductions up to 20% on the strength of a concrete wet than a dry one.
- Presence of chemical phenomena such as *carbonation* of the concrete, that are able of causing a hardening of the surface layer and also determine an overestimation of 50% of the value of compressive strength;
- The rebound index is affected by the movement of the end of the plunger in contact with the concrete. The more the end of the plunger moves, the lower is the rebound;

2.3.6 Interpretation of results

The conversion of rebound index to concrete compressive strength can be achieved by using the calibration chart provided from the manufacturers, or if it is not possible by the production of a calibration graph based on the concrete concerned.

Usually the correlation between the rebound index and the compressive strength of concrete is of this type:

$$R_c = a \cdot (RI)^b$$

Where:

R_c is the concrete compressive strength;

RI is the mean rebound index, after discarding the highest and lowest values;

a, b are two parameters depending on the characteristics of the concrete;

The determination of the experimental calibration curve that relates the compressive strength of concrete and rebound index is achieved by the simultaneous application of non-destructive test and the determination of the mechanical characteristic of concrete by means of compression tests.

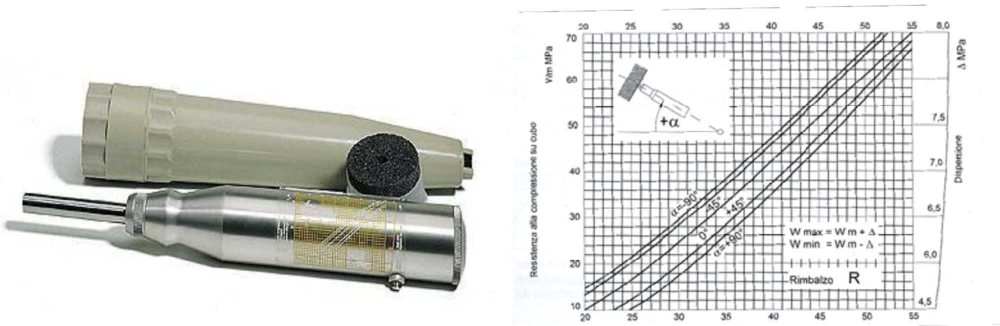


Fig. 2.10. Rebound hammer test equipment and calibration graph.

2.4 Ultrasonic Pulse Velocity test

2.4.1 Generality

The *Ultrasonic Pulse Velocity* (UPV) test is a stress wave propagation method that involves measuring the travel time, over a known path length, of a pulse of ultrasonic waves. The test is based upon the concept to evaluate the speed of transmission (by measuring the time required for the transmission of impulses) of ultrasonic pulses through the investigated element that can subsequently correlated to the compressive strength. Moreover these test can be used to identify the dynamic and static modulus of elasticity, and Poisson ratio and helps to detect the presence of cracks and their length.
The development of this device to measure the pulse velocity happened at the same time in Canada and in England[7]. In 1945 at the U.S. Corps of Engineers, *B.G. Long et al.* [8] studied a test equipment able to measure the speed of a mechanical stress pulse through concrete samples. Their approach involved the simultaneous use of two receivers connected to the material, an particular calibrated hammer and a specially designed timer to measure the time travel of the pulse. The scope of this first attempt was to evaluate the on-site concrete modulus of elasticity. Some years later, *J.R. Leslie and W.J. Cheesman*, two engineers at the Hydro-Electric Power Commission of Ontario (Ontario Hydro) developed an instrument for identifying the presence

Fig. 2.11. UPV test

[7] E.A. Whitehurst, Evaluation of Concrete Properties from Sonic Tests, American Concrete Institute Monograph N. 2, ACI/Iowa State University Press, 1967.

[8] B.G. Long, H.J. Kurtz, and T.A. Sandenaw, *An Instrument and a Technic for Field Determination of Elasticity, and Flexural Strength of Concrete* (Pavements), Journal of the American Concrete Institute, Vol. 16, N. 3, Proceedings Vol. 41, January, 1945, pp 217-231.

of internal cracks, the depth of surface-opening cracks and evaluate the dynamic modulus of concrete. This device was called the *Soniscope* [9] and was employed in the early uses only to measure the pulse velocity, rather than using the latest data for estimating strength or calculating the elastic stiffness. In 1953, *W.E. Parker*[10] reported his attempts to develop for the first time a direct relationships between pulse velocity and concrete compressive strength. In fact he was able to demonstrate his theory showing the presence of distressed concrete by means of several velocity readings. During the same years, at the Road Research Laboratory (RRL) in England *R. Jones* developed an ultrasonic testing apparatus in order to test the quality of concrete pavement, also called: *ultrasonic concrete tester*[11]. Jones was the first to establish the actual problems in using the pulse velocity to estimate concrete strength. Thenceforth numerous researchers has tried to establish correlations between pulse velocity and strength, with different results. Among these we must remember the research of *E.A. Whitehurst* [12](1967) with regards to the classification of concrete quality by means of pulse velocity (see *Tab. 2.2*).

Pulse velocity, m/s	***Condition***
Above 4570	Excellent
3660 to 4570	Generally good
3050 to 3660	Questionable
2130 to 3050	Generally poor
Below 2130	Very poor

Tab. 2.2. Table published by Whitehurst for concrete classification based upon pulse velocity

[9] J.R. Leslie, and W.J. Cheesman, *An Ultrasonic Method of Studying Deterioration and Cracking in Concrete Structures*, Journal of the American Concrete Institute, V. 21, N. 1, Sept., 1949, Proceedings V. 46, pp. 17-36.

[10] W.E. Parker, *Pulse Velocity Testing of Concrete*, Proceedings, American Society for Testing Materials, Vol. 53, 1953, pp.1033-1042.

[11] R. Jones, *The Non-destructive Testing of Concrete*, Magazine of Concrete Research, N. 2, June, 1949, pp. 67-78.

[12] E.A. Whitehurst, *Evaluation of Concrete Properties from Sonic Tests*, American Concrete Institute Monograph, N. 2, ACI/Iowa State University Press, 1967.

2.4.2 Normative references

After the publication of Whitehurst landmark paper, regarding the development of the ultrasonic pulse velocity method, a flurry of interest occurred worldwide, and researches were begun toward developing test standards.
In U.S.A. *J.R. Leslie* proposed a standardized test method firstly in 1955, but it was not until 1967 that it finally became operative with ASTM C597-67T. In the same years in Europe the International Union of Testing and Research Laboratories for Materials and Structures (RILEM) organized a research program on nondestructive testing under the supervision of R. Jones. Finally in 1969, draft recommendation for testing concrete by the ultrasonic pulse velocity method were published (*R. Jones* and *I. Fãcãoaru*[13]).

- *BS 4408: Part.: 1971* Non-destructive Methods of Test for Concrete Measurement of the Ultrasonic Pulses Velocity in Concrete;
- *ASTM C597 – 09* Standard Test Method for Pulse Velocity Through Concrete;
- *UNI EN 12504-4:2004* Testing concrete in structures – Part. 4: Non destructive testing – Determination of ultrasonic pulse velocity;
- *ACI Committee 228: 2003* In-place methods for determination of strength of concrete, Technical Committee Doc. 228.1R-03, American Concrete Institute;
- *UNI EN 1992-1-1:2005* Eurocode 2;
- *UNI EN 13791: 2007* Assessment of in-situ compressive strength in structures and precast concrete.

[13] R. Jones, and I. Fãcãoaru, *Recommendations for Testing Concrete by the Ultrasonic Pulse Method*, Materials and Structures (RILEM), Vol. 2, N. 10, July-August, 1969, pp.275-284.

2.4.3 Scope

"*This test method covers the determination of the propagation velocity of longitudinal stress wave pulses through concrete. This test method does not apply to the propagation of other types of stress waves through concrete*."[14] The applications of the ultrasonic pulse velocity method in testing concrete are based on the fact that a variation of its mechanical characteristics or the presence of both defects (i.e. voids, honeycombs, cracks, etc. ...) and of materials of different elastic characteristics (i.e. reinforcement rods, plastic pipe or any foreign bodies) results in variations in the propagation velocity of the pulses. Consequently, from the detection of: i) phenomena of reflection, refraction, standing waves induced; ii) transit times along certain trajectories, suitably selected or identified; iii) attenuation of vibration energy; it can be made both qualitative and quantitative assessment on concrete. Among these, of particular interest are the evaluations that relate to:

- degree of homogeneity of the concrete;
- concrete gripping characteristics of the first hours;
- casting defects;
- presence of any damaged layers, cracks inside the element and the extent of their depth;
- effectiveness of types of structural rehabilitation (i.e. injections of cement mixtures);
- estimation of the compressive strength of concrete through appropriate calibrations with resistance values obtained from core samples and compressive tests;
- In combination with other methods (i.e. combined method *SonReb*) for estimating the actual on-site strength of the hardened concrete.

[14] ASTM. 2002a. *Standard Test Method for Pulse Velocity Through Concrete*, ASTM C 597. ASTM International, West Conshohocken, PA.

2.4.4 Test equipment and applications

The test equipment (*Fig. 2.12.*) consists of a *pulse generator*, a pair of *transducers* (transmitter and receiver), an *amplifier*, a *time measuring circuit*, a *time display unit*, *connecting cables* and *material of coupling*[15].

- *Pulse Generator*
  The pulse generator shall consist of circuitry for generating pulses of voltage. The latter one affects the transducer power output and the maximum penetration of the longitudinal stress waves. Voltage pulses of 500 to 1000 V have been used successfully. The pulse generator shall produce repetitive pulses at a rate of at least 3 pulses per second. The time interval between pulses shall exceed the decay time for the transmitting transducer. A triggering pulse shall be produced to start the time measuring circuit.

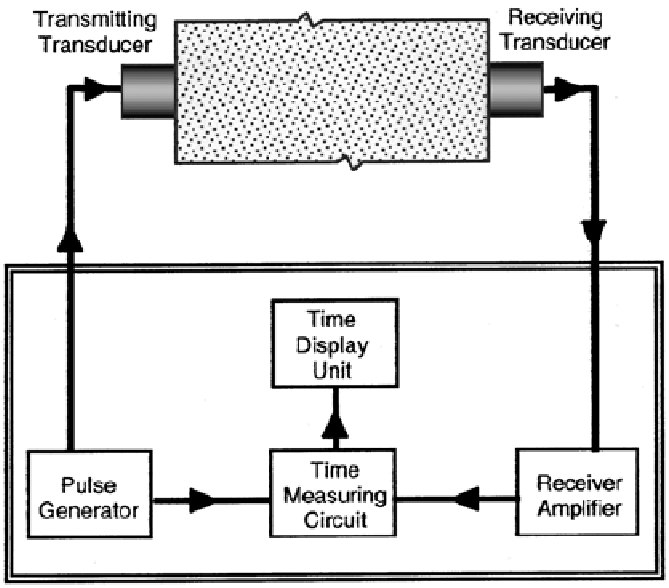


Fig. 2.12. Schematically representation of Pulse Velocity Apparatus (ASTM C597)

[15] *See note 14.*

- *Transmitting and Receiving Transducer*
  The transducer for transforming the electronic pulses into wave bursts of mechanical energy shall have a resonant frequency in the range from 20 to 100 kHz. The transducer shall be constructed of piezoelectric or other voltage-sensitive material, and housed for protection. The receiving transducer shall be similar to the transmitting one. The voltage generated by the receiver shall be amplified as necessary to produce triggering pulses to the time-measuring circuit.

- *Amplifier*
  The amplifier shall have a flat response between one half and three times the resonant frequency of the receiving transducer.

- *Time-Measuring Circuit*
  The time-measuring circuit shall be capable of providing an overall time-measurement resolution of at least *1 μs*. This device shall provide an output when the received pulse is detected, and this output shall be used to determine the transit time displayed on the time-display unit. The time-measuring circuit shall be insensitive to operating temperature in the range from 0 to 40°C.

- *Display Unit*
  A display unit shall indicate the pulse transit time to the nearest *0.1 μs*. In some cases the display unit is equipped with a dedicated field able to bring the value of pulse propagation velocity if it was previously inserted the value of the distance between transducers.

- *Connecting Cables*
  Where pulse-velocity measurements on large structures require the use of long interconnecting cables, use low-capacitance, shielded, coaxial cables.

- *Material of Coupling*
  A viscous material (such as oil, petroleum jelly, water soluble jelly, etc. ...) must be used to ensure efficient transfer of energy between the concrete and the transducers. The aim of this element is to eliminate air between the contact surfaces of the transducers and the concrete.

The test is performed generating by means of the pulse generator a series of pulses. These are introduced into the concrete by means of a transmitting transducer (characterized by the presence of a piezoelectric layer) and a similar transducer acts as a receiver to monitor the arrival of the pulse. A timing circuit is used to measure the travel time of the pulse subsequently reported on the display unit. If it is known the distance between the transducers and set into the device, it is also possible to automatically calculate the pulse velocity. The presence of low density, or cracked, concrete increases the travel time and consequently produce a decrease in the pulse velocity. By conducting tests at various points on a structure, locations with lower quality concrete or damage zone can be identified by their lower pulse velocity.
Although the direction in which the maximum energy is propagated is at right angles to the face of the transmitting transducer, it is possible to detect pulses which have travelled through the concrete in some other direction. It is therefore possible to make measurements of pulse velocity by placing the two transducers on opposite faces (*direct transmission*), or on adjacent faces (*semi-direct transmission*), or the same face (*indirect* or *surface transmission*) of a concrete structure or specimen (*Fig. 2.13.*).

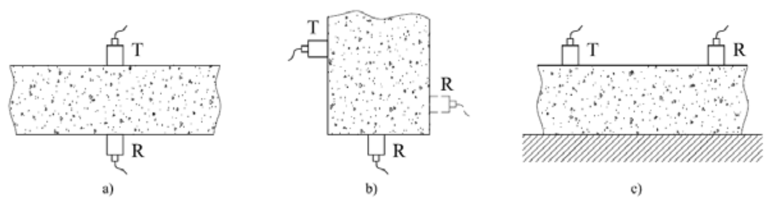


Fig. 2.13. Schematically representation of transducers positioning: a) direct transmission; b) semi-direct transmission; and c) indirect or surface transmission.

2.4.5 Principal drawbacks

In order to provide a correct measurement of ultrasonic pulse velocity which depends essentially on the properties of the concrete under test, it is necessary to consider different factors which are able to influence pulse velocity and its correlation with mechanical and deformability characteristics of the concrete. In practice the test is dependent upon the following aspects:

- The *moisture content* has a strong effects on the pulse velocity, both for chemical and physical aspects. These effects are important in the production of correlations for the estimation of concrete strength and regards mainly the effect of different curing conditions on the hydration of the cement;

- The *path length* over which the pulse velocity is measured should be long enough not to be significantly influenced by the heterogeneous nature of the concrete. It is usually recommended that the minimum path length should be 100 mm for concrete in which the nominal maximum size of aggregate is 20 mm or less and 150 mm for concrete in which the nominal maximum size of aggregate is 20÷40 mm.

- The presence of *steel reinforcement* can alter the ultrasonic pulse velocity. In fact, when possible, measurements in close proximity to steel reinforcing bars, parallel to the direction of pulse propagation should be avoided.

- The presence of *cracks and voids* can reduce considerably the ultrasonic pulse velocity. As a matter of fact, when an ultrasonic pulse travelling through concrete meets a concrete-air interface, there is negligible transmission of energy across this interface. Thus, any air-filled crack or void lying immediately between two transducers will obstruct the direct ultrasonic beam when the projected length of the void is greater than the width of the transducers and the wavelength of sound used.

2.4.6 Interpretation of results

The fundamental non-destructive parameter that it can be determined with the application of this test type is the ultrasonic pulse velocity ***V*** [m/s]. This can be calculated as follows:

$$V = L/T$$

Where:

V	is the pulse velocity, [m/s];
L	is the distance between centers of transducer faces [m];
T	is the transit time [s].

Even if a direct unique relationship between concrete compressive strength and ultrasonic pulse velocity does not exists, under specified conditions some information on its quality can be derived (*Tab. 2.3.*).

Ultrasonic Pulse velocity [m/s]	***Quality of Concrete***
> 4500	Excellent
3500 to 4500	Good
3000 to 3500	Doubtful
2000 to 3000	Poor
< 2000	Very poor

Tab. 2.3. Quality of concrete as a function of the ultrasonic pulse velocity

2.4.7 Concrete compressive strength evaluation by means of UPV

From the analysis of the scientific literature in the field of ultrasonic investigations, there are numerous experimental formulations able of correlating the concrete compressive strength with the measurement of the non-destructive parameter of the ultrasonic pulse velocity. Therefore between the numerous correlations proposed to limit the uncertainty of this method, three of the most reliable are taken into account, that are the formulas proposed by: i) *Hisham, Y. Qasrawi*[16]; ii) *R. Giannini*[17] and iii) *M. Bilgehan, P. Turgut*[18].

Eq. A: (*Y. Hisham-Qasrawi*, 2000)

$$f_c = 1,88 \cdot 10^{-21} \cdot (V)^{6,184} \qquad \text{[MPa]}$$

Eq. B: (*R. Giannini*, 2003)

$$f_c = (-307 + 0,157 \cdot V)/10 \qquad \text{[MPa]}$$

Eq. C: (*M. Bilgehan, P. Turgut*, 2010)

$$f_c = 0,8822 \cdot e^{0,0008 \cdot V} \qquad \text{[MPa]}$$

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[16] Y.H. Qasrawi in: *Cement and Concrete Research*, Vol. 30, pp. 739-746 (2000).

[17] R. Giannini et al. in: *10° Congresso Nazionale AIPnD*, Ravenna, Italy (2003).

[18] M. Bilgehan, P. Turgut, *Artificial Neural Network Approach to Predict Compressive Strength of Concrete through Ultrasonic Pulse Velocity*, Research in Nondestructive Evaluation, 21:1, 1-17, 2010.

2.5 Combined method SonReb

2.5.1 Generality

The poor reliability of rebound hammer and ultrasonic pulse velocity due to different aspects could be partially contrasted by using both methods together. One of the most employed NDT combined methods in practice is the SonReb method, described in RILEM 7 NDT and 43 CND. This technique achieves an improvement of the accuracy by the use of various correction factors taking into account the influence of different concrete mixture proportion. Hence, since 1950s, efforts were dedicated to develop combined methods, in which the results of more nondestructive tests are used to estimate concrete strength. The most important aspect of this approach is that if two methods are influenced in different ways by the same factor (i.e. the increasing moisture content that increases pulse velocity but decreases the rebound number), their combined use can produces a cancelling effect that improves the accuracy of the estimated strength.
Among the various attempts, the combination of ultrasonic pulse velocity and rebound index have received the most consideration. In 1970 Dr. I. Făcăoaru[19] of the Building Research Institute in Romania developed an indirect method that combined the results obtained from the rebound hammer and the ultrasonic pulse velocity test with the concrete mechanical characteristic. In fact by using multiple-regression analysis, the compressive strength (f_c) of standard core specimens is expressed as a function of the average rebound index (*IR*) and the average ultrasonic pulse velocity (*V*). This NDT methods became known as the *SonReb method* (the name SonReb derives from the conjunction of the terms SONic for the UPV test and REBound from the Rebound Hammer test).

[19] I. Făcăoaru, *Non-destructive Testing of Concrete in Romania*, Proceedings, 1970, Symposium on Non-destructive Testing of Concrete and Timber, 11-12 June, 1969, Institution of Civil Engineers, pp. 39-49

2.5.2 Regression analysis

The combined method SonReb can evaluate the concrete compression strength by combining the experimentally obtained non-destructive parameters with correlations as follow:

$$f_c = f_0 \cdot e^a \cdot V^b \cdot (RI)^c$$

Where:

f_c	is the concrete compression strength, [MPa];
f_0	is the units conversion factor, [usually f_0 = 1 MPa·s/m];
V	is the ultrasonic pulse velocity [m/s];
RI	is the rebound index;
a,b,c	are dimensionless correlation parameters to be determined with regression analysis.

The previous relation can be changed using logarithmic notation, in the multiple linear regression analysis:

$$Ln(f_c) = a + b \cdot Ln(V) + c \cdot Ln(RI)$$

To identify the parameters of the linear regression is usually applied the method of *minimum weighted squares*. Assumed that w_i represents the known weight to be attributed to the standard deviation, its value depends on the relation:

$$w_i = f_{c,i}^2$$

The *mean square error* of the linear regression is obtained by the following expression:

$$\varepsilon^2 = \frac{1}{n}\sum_{i=1}^{n} w_i \cdot \left[Ln\left(f_{c,i}\right) - \left(a + b \cdot Ln\left(V_i\right) + c \cdot Ln\left(RI_i\right)\right)\right]^2$$

The unknown parameters a, b, c are determined imposing the condition that they minimize the mean square error. The condition of minimum is detected by comparing to zero the derivatives of the function ε^2 with unknown parameters, obtaining the system of equations:

$$\sum_{i=1}^{n} w_i \cdot Ln\left(f_{c,i}\right) = a \cdot \sum_{i=1}^{n} w_i + b \cdot \sum_{i=1}^{n} w_i \cdot Ln\left(V_i\right) + c \cdot \sum_{i=1}^{n} w_i \cdot Ln\left(RI_i\right)$$

$$\sum_{i=1}^{n} w_i \cdot Ln\left(f_{c,i}\right) \cdot Ln\left(V_i\right) = a \cdot \sum_{i=1}^{n} w_i \cdot Ln\left(V_i\right) + b \cdot \sum_{i=1}^{n} w_i \cdot Ln^2\left(V_i\right) +$$
$$+ c \cdot \sum_{i=1}^{n} w_i \cdot Ln\left(RI_i\right) \cdot Ln\left(V_i\right)$$

$$\sum_{i=1}^{n} w_i \cdot Ln\left(f_{c,i}\right) \cdot Ln\left(RI_i\right) = a \cdot \sum_{i=1}^{n} w_i \cdot Ln\left(RI_i\right) +$$
$$+ b \cdot \sum_{i=1}^{n} w_i \cdot Ln\left(V_i\right) \cdot Ln\left(RI_i\right) + c \cdot \sum_{i=1}^{n} w_i \cdot Ln^2\left(RI_i\right)$$

From the resolution of this system of equations, the values of *a*, *b* and *c* are determined.

2.5.3 Most reliable formulation

During the years, several correlation equations have been developed and presented by numerous authors. In this study, some of the most reliable and employed formulations available in literature are considered:

Eq. A: (*R. Giacchetti, L. Lacquaniti*[20])

$$f_c = 7{,}695 \cdot 10^{-11} \cdot (RI)^{1{,}4} \cdot (V)^{2{,}6} \qquad \text{[MPa]}$$

Eq. B: (*A. Di Leo, G. Pascale*[21])

$$f_c = 1{,}2 \cdot 10^{-9} \cdot (RI)^{1{,}058} \cdot (V)^{2{,}446} \qquad \text{[MPa]}$$

Eq. C: (*J. Gasparik*[22])

$$f_c = 0{,}0286 \cdot (RI)^{1{,}246} \cdot (V)^{1{,}85} \qquad \text{[MPa]}$$

The main distinction between this formulations lies especially in the different relevance given to the correction factors and the two parameters determined onsite: the rebound index (RI) and the virtual ultrasonic pulse velocity (V). For this reason it seems to be interesting to take into account also the results obtained by combining two or more expressions (Technical Standards of Toscana Region).

[20] R. Giacchetti, L. Lacquaniti, *Controlli non distruttivi su impalcati da ponte in calcestruzzo armato*, Technical Note 04, 1980, University of Ancona, Faculty of Engineering, Institute of Science and Technology.

[21] A. Di Leo, G. Pascale, *Prove non distruttive sulle costruzioni in cemento armato*, Conference on Non Destructive Testing and Quality System for Reliability and Safety of Civil Structures, Bologna, SAIE'94, October, 21, 1994.

[22] J. Gasparik, *Prove non distruttive in edilizia*, A.I.P.N.D., Brescia, 1992

Chapter 3
Artificial Neural Network (ANN)

Sommario

Le reti neurali artificiali sono sistemi di elaborazione dell'informazione che cercano di simulare in un sistema informatico il funzionamento dei sistemi nervosi biologici che sono costituiti da un elevato numero di neuroni connessi tra loro in una complessa rete. Ogni neurone è collegato mediamente con una decina di migliaia di altri neuroni. Si hanno quindi centinaia di miliardi di connessioni. Il comportamento intelligente emerge dalle numerose interazioni tra le unità interconnesse. Alcune di queste unità ricevono informazioni dall'ambiente (unità d'ingresso: input layer), altre emettono risposte nell'ambiente (unità di uscita: output layer) e altre ancora comunicano solamente con le unità all'interno della rete (unità nascoste: hidden layer). Il legame input–output, ovvero la funzione di trasferimento della rete, non viene programmato ma è semplicemente ottenuto da un processo di apprendimento basato su dati empirici.
Lo studio delle reti neurali risale ai primi tentativi di tradurre in modelli matematici i principi dell'elaborazione biologica trova la prima applicazione in ambito informatico sin dagli anni quaranta.
Esistono diversi tipi di reti neurali artificiali (ANN) in relazione alla tipologia di connessioni che legano i neuroni dei diversi strati, alle funzioni di attivazione e agli algoritmi di apprendimento. A seconda di come sono le

connessioni tra neuroni artificiali si distinguono le tre classi principali di ANN: *reti feed-forward*, *reti cellulari* e *reti feed-back*.
Uno degli aspetti più importanti delle reti neurali riguarda il loro processo di apprendimento. Questo può essere di tre tipi: i) supervisionato (*supervised learning*); ii) non Supervisionato (*unsupervised learning*); iii) rinforzato (*reinforcement learning*). Da un punto di vista matematico l'apprendimento di una rete neurale consiste nella *ricerca di un minimo di una funzione in uno spazio n-dimensionale*. Tale funzione è data dalla variazione dell'errore in base ai pesi della rete. L'algoritmo maggiormente impiegato nell'apprendimento con supervisione è l'algoritmo di *backpropagation error*, il quale attraverso la modifica dei pesi delle connessioni minimizza l'errore totale della rete

3.1 Introduction

Artificial Neural Networks (ANN) are information processing systems that try to simulate in a computer program the behavior of biological nervous systems which are constituted by a large number of neurons connected together in a complex network. Each neuron is connected with an average of about ten thousand other neurons. Therefore, there are hundreds of billions of connections. The intelligent behavior arises from interactions among numerous interconnected units. Some of these units receive information from the external environment (i.e. *input layer*), others emit responses in the environment (i.e. *output layer*) and still other communicate only with other units inside the network (i.e. *hidden layer*). The input-output ratio, i.e. the transfer function of the network, is not programmed but is simply obtained by a learning process based on empirical data.
The study of neural networks dates back to early attempts to translate the biological principles of drafting into mathematical models. Their first application developed since the 1940s.

There are several types of artificial neural networks (ANN) in relation to the type of connections that link the neurons of the different layers, to the activation functions and learning algorithms. Depending on the type of connections between artificial neurons, it can be distinguish the three main classes of ANN: *feed-forward networks*, *cellular networks* and *feed-back networks*.One of the most important aspects of neural network is their learning process. This can be of three principal types: i) *supervised learning*; ii) *unsupervised learning*; iii) *reinforcement learning*. From a mathematical point of view the learning of a neural network consists in the research for a minimum of a function in an n-dimensional space. This function is given by the variation of the error based on the *weights* of the network. The algorithm used in the learning with more supervision is the *backpropagation error algorithm*, which minimizes the total error of the network through the modification of the *weights* of the connections.

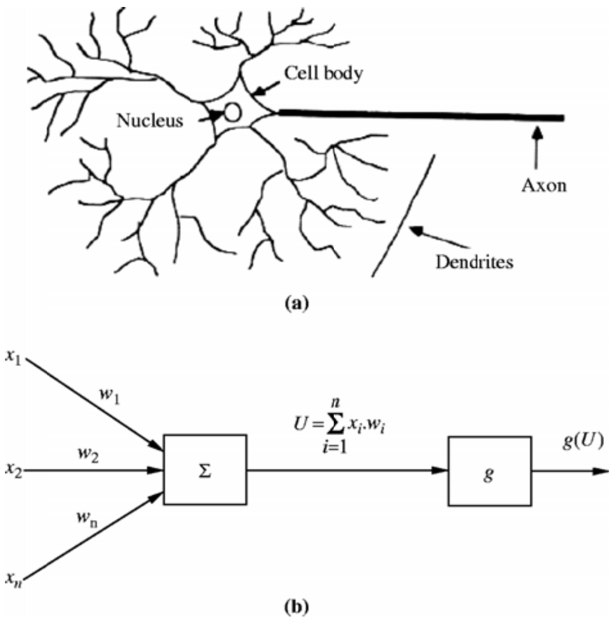

Fig. 3.1. Schematic representation of a biological neuron (a) and artificial neural network (b)

3.2 State of the art

The interest in the Artificial Neural Networks arises in the early 1943, when *W.S. McCullock* and *W.A. Pitts*[1] proposed a mathematical model of a single artificial neuron. They formulate the *cybernetic approach* based on the development of the first neural network model. Towards the end of 1940s, *D. Hebb*[2] explained the mechanism of "*collecting information*" by the biological neural networks, which is a key element to establish the interconnection between the artificial neurons. He introduced for the first time a learning rule based on the relationship between the output of different neuron and a learning factor. At the same time, *K.S. Lashley* and *S.R. Cajal*'s studies on the human brain indicated that the organization of knowledge and memory were based on distributed representations. In 1958, *F. Rosenblatt*[3] has developed an artificial neural network system, also called as *Perceptron* (*Fig. 3.2*). This network has been designed with two separate layers of neurons, i.e. the input and output layer. The output layer neurons obtained signals from the input layer neurons, while neurons of the same layer do not communicate each other mutually. A major achievement was to prove the *theorem of convergence of perceptron*, which guarantees a finite number of iterations of the learning process of neural network.

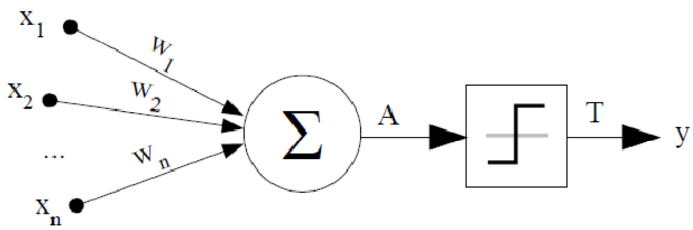

Fig. 3.2. Scheme of a Perceptron and its operation system.

[1] W.S. McCullock, W.A. Pitts, *A logical calculus of ideas immanent in nervous activity*, Bullettin of Mathematical Biophysics. Vol. 5, 1943.

[2] D. Hebb, *The Organization of Behavior*, Wiley, New York, 1949.

[3] F. Rosenblatt, *Principles of Neurodynamics: Perceptrons and the Theory of Brain Mechanism*, Spartan Books, Washington D.C., 1962.

The Perceptron is a neural network composed of logical devices capable of solving mainly problems related to simple pattern recognition. It can be considered nowadays the main prototype of the more complex structures that were developed later. In 1954 *B. Gabor* invented the "*learning filter*" that uses gradient descent to obtain optimal weights that minimize the mean squared error between the observed output signal and a signal generated based upon the network information.

Another major contribution to the development of these neural systems occurs in 1960 by *B. Widrow* and *M. Hoff*[4]. These American researchers built the first commercial neurocomputer, made up of innovative elements and dynamics of learning, called *Adaline* (*Ada*ptive *line*ar element), and the network was called *Madalina*. It seemed then for the first time that the construction of *artificial brain* could be possible and composed only of large networks of artificial neurons built properly.

In 1969 *M. Minsky* and *S. Papert*[5], researchers at the Massachusetts Institute of Technology, publish an highly critical analysis of the Perceptron type machines. They demonstrated mathematically the limitations of the neural networks in solving different problems (i.e. in the determination of parity of a binary number, the calculation of an XOR function of 2-bit or the classification of the images according to their connectivity). Nevertheless these problems could be solved only by neural networks in which each neuron is connected with all the other neurons of the network: in such a system, the number of connections grow exponentially with increasing the number of neurons, contrary to what occurs in biological systems in which the connections grow linearly.

After the publication of *M. Minsky* and *S. Papert*'s theories, many researchers have give up the study on neural networks, who turned to the field of Artificial Intelligence (AI), an approaches rival and apparently more

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[4] B. Widrow, M. Hoff, *Adaptive switching circuits*. In IRE WESCON Convention Record. IRE, New York, Vol. 4, 1960, p. 96-104.

[5] M. Minsky, S. Papert, *Perceptrons – Expanded Edition*. MIT Press, Cambridge, 1969

promising to the ANN. One of the principal reason of this change of interest was caused by the fact that the technology were not enough developed to simulate the complex behavior of neural network.
In the 1970s the search continued with different theoretical contributions and a few applications. Some researchers such as *S. Amari*, *K. Fukushima* and *S. Grossberg*[6] attempted to simulate the behavior of brain neurons with networks of units operating in parallel mode. In addition formulated mathematical theories and architectures to identify and classify the characteristic features of the forms to be recognized and to build the first associative memories.
Important results were achieved between the late 70's to mid 80's with the development of a multi-layer network and its learning algorithm, now better known under the name of the *Backpropagation error algorithm*. This algorithm has been developed by *D. Rumelhart et al*[7], after several attempts of many other researchers. Due to the innovations of this learning algorithm, many of the restrictions on the applications of ANNs indicated by M. Minsky and S. Papert, have been removed.
A great contribution to the development of artificial neural networks was also provided by *T. Kohonen* that has developed a network of associative memory, and *J. Hopfield*[8] networks with the introduction of feed-back, as well as of the networks ART (Adaptive Resonance Theory).

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[6] S. Grossberg, *Classical and instrumental learning by neural networks*, Progress in Theoretical Biology (3): 51-141, 1974.

[7] D. Rumelhart, G. Hinton, and R. Williams, *Learning internal representations by error propagation*. In Parallel Distributed Processing, D. Rumelhart and J. McClelland, Eds. Vol. 1 MIT Press, Cambridge, Chapter, Vol. 8, 1986, p. 318-362.

[8] J. Hopfield, *Neural networks and physical systems with emergent collective computational abilities*, Proceedings of the National Academy of Sciences, USA, Vol. 79, 1982, 2554-2558.

3.3 Structure of artificial neural network

There are many types of neural networks that are differentiated on the basis of some basic characteristics: a) type of use; b) the activation function; c) type of learning (and as a consequence the *learning algorithm*); and d) architecture of connections.
From the point of view of the *type of use* we can distinguish three basic categories:

- *associative memories*: neural network that can learn associations between different patterns (complex set of data, i.e. the pixels of an image) so that the presentation of a pattern α gives as output the pattern β, even if the pattern α is totally or partially imprecise (resistance to noise).
- *simulators of mathematical functions*: they are able to understand the function that links the output with the input according to the examples given (*target* data) in the learning phase. After the learning phase, the network is able to provide an output in response to an input also different from those used in the training examples. This means a capacity of the network of interpolation and extrapolation on the data of the training set. This capacity can be easily verified by training a network with a sequence of data input / output from a known function and is, instead, just useful for the treatment and the prediction of phenomena of which it is not mathematically defined the link between input- output. In any case the network behaves as a *black box*, since it does not reveal in legible the transfer function of which is contained inside.
- *classifiers*: neural networks that make possible to classify the data into specific categories based on similarity of characteristics. In this last type of network there is no concept of *unsupervised learning* (see *Paragraph 3.5.*), in which the input data are distributed on categories

that are not predefined. The learning algorithm of a neural network dependent on the type of use of the same, as well as the architecture of the connections.

As regards the types of activation functions and the categories of learning, they will be briefly exposed respectively in paragraphs 3.4. and 3.5. of this chapter.
The basic classification to define its structure is certainly that relating to the architecture of the connections. From the point of view of the type of connections between neurons we can distinguish three basic categories, which in turn comprise other types (as shown in *Fig. 3.3.*). They are: i) Feed-Forward Neural network (FFN); ii) Cellular Neural Network (CNN); iii) Feed-Back Neural network (FBN).

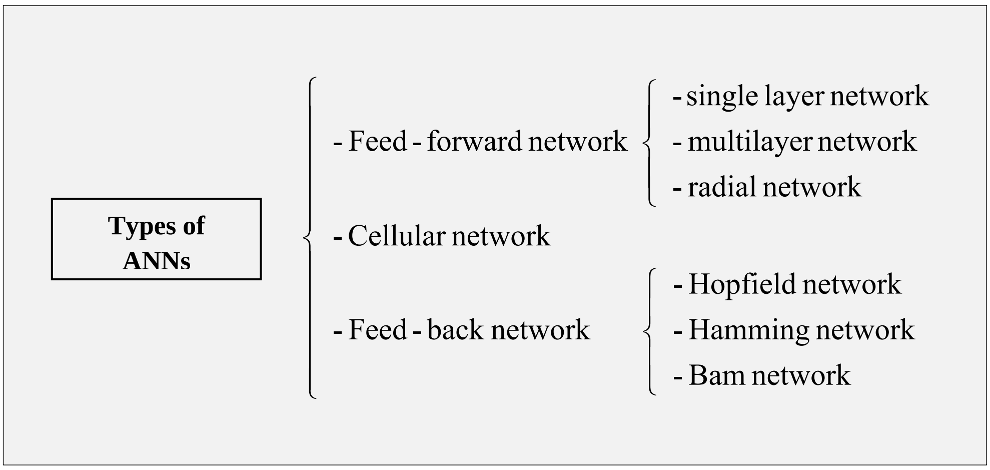
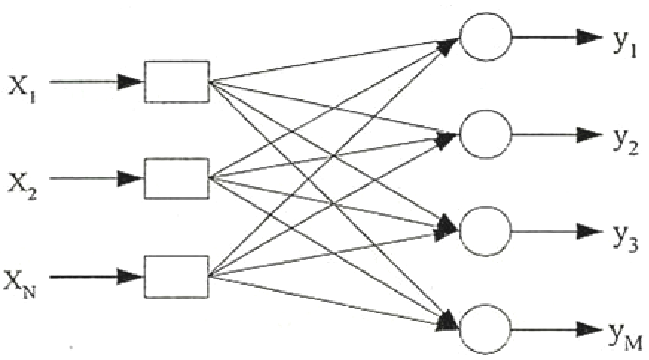


Fig. 3.3. Scheme of the different types on ANNs

3.3.1 Feed-Forward Network (FFN)

Feed-Forward Network (FFN) is the simplest and most used typology and is constituted by more than two layers of neurons. In other words to the input and output layer is supplemented by one or more hidden layers. In this type of neural network each neuron is connected to all the neurons of the previous layer but has no connection with the neurons of its own layer and the signal propagates in a unidirectional way from input to output through the hidden layer.
The way in which the signal is processed varies depending on the specific architecture implemented. These networks are generally used in supervised learning methods and training (i.e. *Backpropagation error algorithm*).
An example of this type of neural network is the *Radial Basis Function* (RBF). The neural network RBF is a particular architecture of feed forward networks with a single hidden layer, in which the activation function for output is defined as the identity function, then the deterministic part of the model that relates the x-inputs to the y-output is a linear combination of *n* basis non-linear functions (*Fig. 3.4.*).



With:

$$x = (x_0, x_1, ..., x_N)$$
$$y = (y_0, y_1, ..., y_N)$$

x = input vector

y = output vector

Fig. 3.4. Schematic representation of a Feed-forward Neural Network.

3.3.2 Cellular Neural Network (CNN)

Cellular Neural Networks (CNN) are characterized by the fact that individual neurons are connected only locally with their neighbours. These connections are bidirectional and affect neurons in the immediate vicinity. They are described by nonlinear differential equations and their natural field of application is related to image processing in real time. In fact, the parallel architecture makes such networks are particularly suitable for real-time applications, while the two-dimensional structure makes them suitable to the processing of the images. The general structure of the CNN has been presented in *Fig. 3.5*. In the figure the arrangement of neurons is characterized by *N-rows* and *M-columns*. The fundamental difficulty in applying this type of network, due to their complexity, is to develop efficient methods, effective and universal for the design of their structure and methods of learning. These networks usually apply methods of unsupervised learning (see Section 3.5). In this way the objective of the analysis is not previously placed in the form of specific examples and the only information available are the correlations between the input data of the network. The network analysis is to create "categories" based on correlations of the input signals to generate the corresponding categories of output signals.

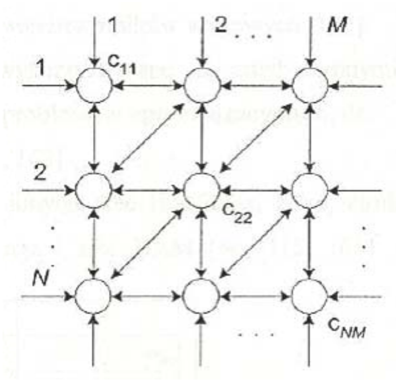
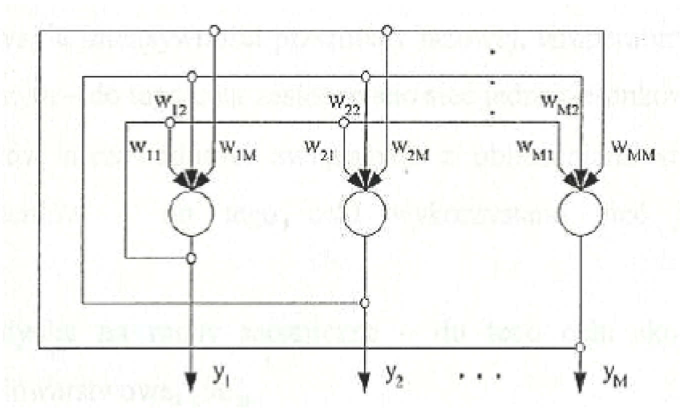


Fig. 3.5. Schematic representation of a Cellular Neural Network.

3.3.3 Feed-Back Network (FBN)

Feed-Back Networks (FBN) have connections between different neurons directly or indirectly in relation to the recursive single unit or to the layer which belong to the unit: in this case the output determined by the network is partially reinstated in the system at the end of a succession of iterations as the increment input thereby creating a feedback circuit (*Fig. 3.6.*).
From training examples FBNs can learn to map input sequences to output sequences. In principle they can implement almost arbitrary sequential behavior. In some field of applications, Feed-back networks are biologically more plausible and computationally more powerful than other adaptive models such as Feed-forward and Cellular networks.
They can be used to solve efficiently many previously unlearnable tasks, as: i) recognition of temporally extended patterns in noisy input sequences; ii) derivation of information conveyed by the temporal distance between events; iii) analysis of aspects regarding speech segmentation and speech recognition.
One of the most widespread feed-back networks is the Hopfield's network (usually used in the systems of reproduction).



With:

$y = (y_0, y_1, ..., y_N)$

W_{ij}

y = output vector

W_{ij} = synaptic weigth

Fig. 3.6. Schematic representation of a Feed-Back Neural Network.

3.4 Activation Function

The fundamental building blocks of each neural networks, both biological and artificial, are represented by the neurons. These can be seen as an elementary information processing units characterized by connections (synapses) w_{ij} able to transfer the signal (stimulus) a_j into other neurons. The synaptic weights w_{ij} can be seen as a value that represents the different importance of its input.
Each neuron sums the weighted inputs $w_{ij} \cdot a_j$ emitting an y-output which varies according to the specific activation function choice.
Thus, in order to better understand the operation of a neural network it is necessary to introduce the concept of activation function in mathematical terms, that is:

$$a_i = f\left(\sum_j w_{ij} \cdot a_j\right)$$

Where: $\boldsymbol{a_{ij}}$ is the activation value;
$\boldsymbol{w_{ij}}$ is the synaptic weight;

Therefore, this transfer function allows the receiving neuron to transmit the received signal by modifying it.
In literature there are many different types of activation functions. Between them, the most commonly used for computational analysis are:

- *Linear function*

$$f(x) = ax \quad con \quad a > 0$$

- *Thresohold function*

$$f(x) = f(u-x) = \begin{cases} 1 \;\; per \;\; u > \theta \\ 0 \;\; per \;\; u \le \theta \end{cases}$$

- *Sigmoid logistic function*

$$f(x) = \frac{1}{1+e^{(-x)}}$$

- *Hyperbolic tangent function*

$$f(x) = \frac{1-e^{(-x)}}{1+e^{(-x)}}$$

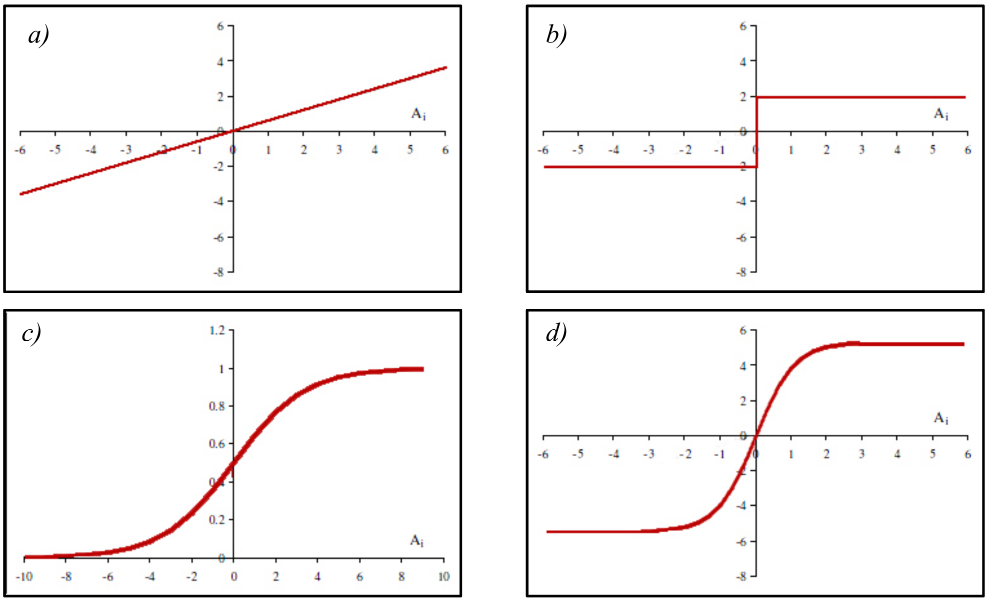


Fig. 3.7. Schematic representation of different activation function: a)Linear function; b) Threshold function; c) Sigmoid logistic function; and d) Hyperbolic tangent function.

3.5 Type of Learning

The application of a neural network system in order to resolve a specific problem the values of synaptic weights represent the first unknown parameter to be determined. To determine their value able to minimize the total error of the network, a process of learning (or training) must be carry out.
This process can be of three main types:

a. *Supervised learning*: when it is possible to use a data set for training includes examples of typical input with its output (in order to permit that the network can "*learn*" the relationship between them). Subsequently, the network is trained by means of a suitable algorithm, which uses such data in order to modify the synaptic weights of the network in such a way as to minimize the prediction error relative to the set of training. If the training is successful, the network learns to recognize the unknown relationship that links the input variables to the output results, and is therefore able to make predictions also in cases where the output is unknown, in other words, the ultimate aim of this learning type is the prediction of the output value for each valid input data, based only on a limited number of examples of correspondence.

b. *Unsupervised learning*: this learning is based on algorithms that modify the synaptic weights of the network by reference to a data set that includes only the input parameters. The unsupervised learning is often used to develop techniques for data compression.

c. *reinforcement learning*: similar supervised learning, from which it differs by the absence of input-output relationship. This can be substitutes with the use of suitable algorithms capable of identifying the best modus operandi.

From a mathematical point of view the learning of a neural network consists in the *search for a minimum of a function in an n-dimensional space.* This function is given by the error variation based on the synaptic weights of the network. Therefore it has to change the value of the various synaptic weights according to our set of examples, strengthening some and weakening other weights. In order to search for a minimum, it is usually used the gradient descent technique, (i.e. make small movements on the function proportional to its derivative and of opposite sign, trying to converge to a minimum, where the derivative being nothing moving stops – *Fig. 3.8.*).

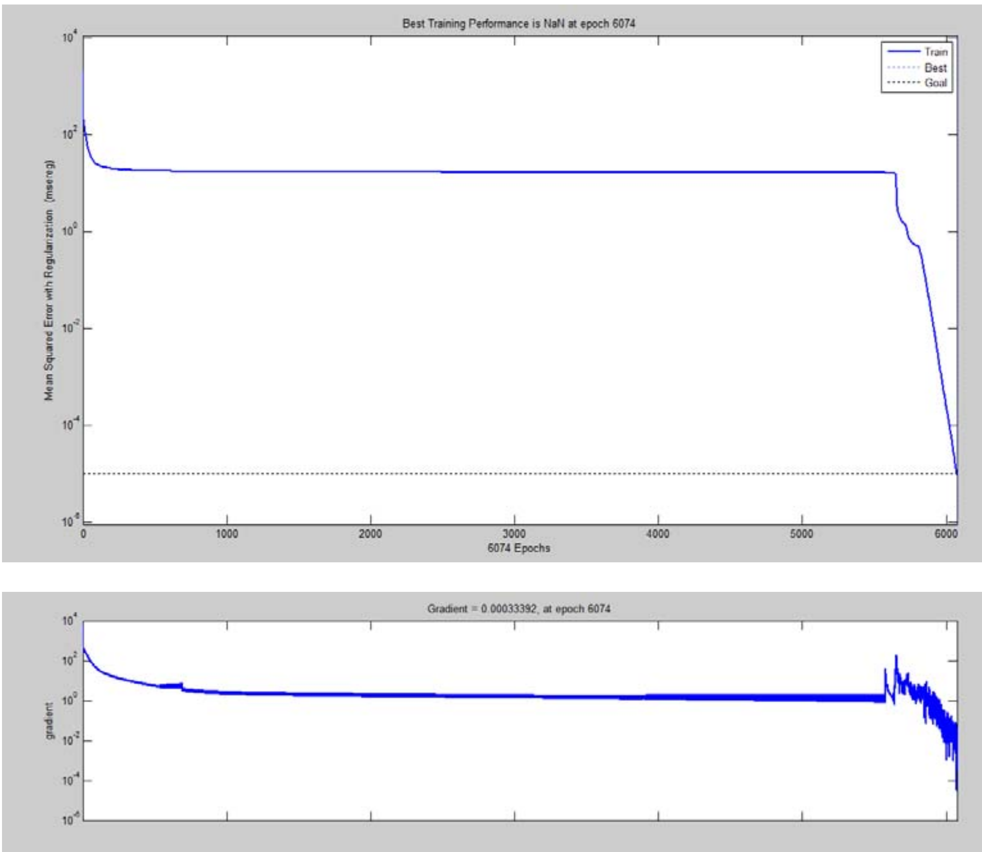


Fig. 3.8. Minimization of total error by gradient descent

3.6 Backpropagation error algorithm (BPE)

The algorithm most used in the supervised learning is the *backpropagation error algorithm* (BPE), which through the modification of the synaptic weights of the connections minimizes the *total error of the network* (E_{tot}).

The error E_i of a generic neuron *i* of the output layer can be defined as:

$$E_i = (y_T - y_O)$$

Where:

y_T : is the *target value*;

y_O : is the *output value*.

Meanwhile, the variation of the synaptic weights w_{ij} of the output layer can be defined as:

$$w_{ij} = w_{ij} + \varepsilon \cdot a_j \cdot \Delta_i$$

Where:

Δ_i : is the *error variation velocity* of the neuron;

ε : is the *learning factor* (usually varies between 0 and 1);

a_j : is the activation value.

With the increase of ε, the variation of synaptic weights will be characterized by large step (bringing the learning process to a continuous oscillation without identifying the best combination that minimizes the error).

With decreasing ε of the variation of the synaptic weights will be characterized by small steps (by determining the minimum error, but at a cost in terms of training time much greater).

The error variation velocity Δ_i is defined in a general form, as follow:

$$\Delta_i = E_i \cdot f'\left(\sum_j w_{ij} \cdot a_i\right)$$

Where:

f' (…) : represents the *derivate of the activate function.*

The error E_j of a generic neuron *j* of the input layer and/or hidden layer can be determined using the following expression:

$$E_j = \sum_i w_{ij} \cdot \Delta_i$$

Thus the total error of the network E_{tot} can be estimated by the equation of the Mean Square Error (MSE):

$$E_j = MSE = \frac{1}{n}\sum_{i=1}^{n}(y_T - y_O)^2$$

Or calculated with the expression of the Root Mean Square Error (RMSE):

$$E_j = RMSE = \sqrt{\frac{1}{n}\sum_{i=1}^{n}(y_T - y_O)^2}$$

Chapter 4

Results and Analysis

Sommario

In questo ultimo capitolo della tesi, due tipologie di reti neurali artificiali (una per l'indagine UPV e una per il metodo SonReb) capaci di stabilire una correlazione non-lineare tra dati input ricavati mediante prove non distruttive (come l'indice di rimbalzo e la velocità di propagazione dell'onda ultrasonora) e dati output (come la resistenza a compressione) sono state sviluppate ed implementate in linguaggio MatLab.

Tra i numerosi tipi di ANN sono state prese in esame reti feed-forward composte da 3 layer (input, output e hidden layer), con funzione di attivazione sigmoidale logistica e apprendimento supervisionato con algoritmo di backpropagation error per minimizzare l'errore attraverso la tecnica di discesa del gradiente. L'efficacia delle reti è stata studiata in funzione dell'unico parametro variabile nella loro architettura: il numero di neuroni presenti nell'hidden layer.

Questo metodo ha permesso di individuare l'efficacia del numero di connessioni tra gli strati della rete in relazione all'effettivo miglioramento in termini di risultato globale.

I risultati ottenuti con le reti neurali artificiali sono poi stati confrontati con quelli conseguiti con i metodi classici delle analisi non-distruttive basate sulle formule di regressione multiparametrica. Data la congruenza dei dati in input tra metodo SonReb/UPV e ANNs (i.e. parametri non-distruttivi di velocità di

propagazione e indice di rimbalzo) e dati output da conseguire (resistenza a compressione del calcestruzzo) è stato possibile strutturare le valutazioni di efficacia sulla base della valutazione della radice dell'errore quadratico medio (RMSE).

4.1 Introduction

In this final chapter of the thesis, two types of artificial neural networks (one for the UPV assessment – see *Fig. 4.1.a.* – and one for the SonReb method – *Fig. 4.1.b.*) have been developed and implemented in *MatLab* language. These networks are able to establish a non-linear correlation between input data obtained using non-destructive tests (such as the Rebound Index *RI* and the ultrasonic pulse velocity *UPV* of elastic waves) and output data (such as the concrete compressive strength).
Among the many types of ANN two feed-forward networks composed of 3 layers (input, output and hidden layers), with *sigmoid logistic activation function* and *supervised learning* with *backpropagation error algorithm* (to minimize the error through the technique of gradient descent) were examined. The effectiveness of the networks has been investigated as a function of the only variable parameter in their architecture: the number of neurons of the hidden layer. This method has allowed to identify the efficiency of the number of connections between the layers of the network in relation to the actual improvement in terms of overall result.

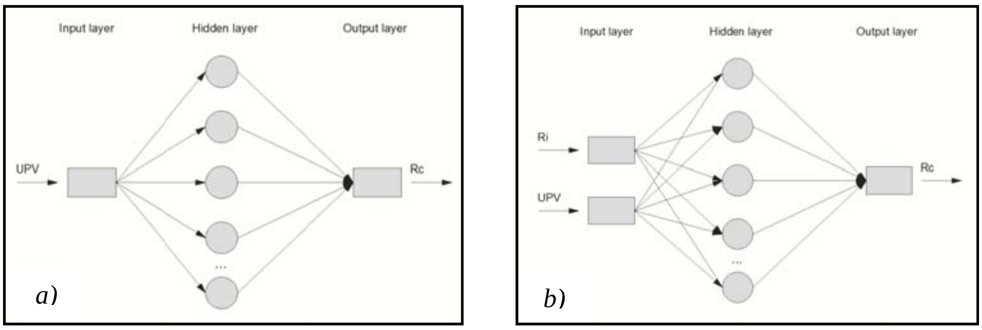


Fig. 4.1. Schematic representation of ANN with only UPV as input data (a) and ANN with UPV and RI as input data (b).

The results obtained with artificial neural networks were then compared with those obtained with classical methods of analysis based on non-destructive multi-parameter regression formulas. Given the congruence of the input data between SonReb/UPV methods and ANNs (i.e. ultrasonic pulse velocity and rebound index) and output data to be achieved (concrete compressive strength) has been possible to structure the feedback on the effectiveness of basis of the assessment of the *Root Mean Square Error* (RMSE).

4.2 Comparison between UPV methods – $ANN_{(UPV)}$

4.2.1 Analysis with UPV methods

Assessments made with the regression formulas found in the literature (see Chapter 2.4.7.) have showed substantially the same result in terms of efficiency. The most effective formulation evaluated in terms of RMSE was that of *R. Giannini*, 2003 (*Eq. B*), as it can be seen as follows.

N.	f_c **[Mpa]**	**UPV [m/s]**	$f_{c\ (UPV\ A)}$ **[Mpa]**	$f_{c\ (UPV\ B)}$ **[Mpa]**	$f_{c\ (UPV\ C)}$ **[Mpa]**
1	10,00	2470	1,80	8,08	6,36
2	12,60	2450	1,71	7,77	6,26
3	17,50	2830	4,17	13,73	8,49
4	17,80	3250	9,81	20,33	11,88
5	18,50	2960	5,50	15,77	9,42
6	18,70	2930	5,17	15,30	9,20
7	18,90	3285	10,48	20,87	12,21
8	20,60	3120	7,62	18,28	10,70
9	23,25	3140	7,93	18,60	10,88
10	25,60	3500	15,51	24,25	14,51
11	27,80	2965	5,56	15,85	9,46
12	29,30	3470	14,71	23,78	14,16
13	32,16	3490	15,24	24,09	14,39
14	36,80	3750	23,77	28,18	17,72
15	54,10	3900	30,29	30,53	19,98
16	56,60	4095	40,96	33,59	23,35
RMSE [Mpa]			14,4278	9,6729	16,3484

Tab. 4.1. Summary of UPV data and correlations for on-site concrete strength

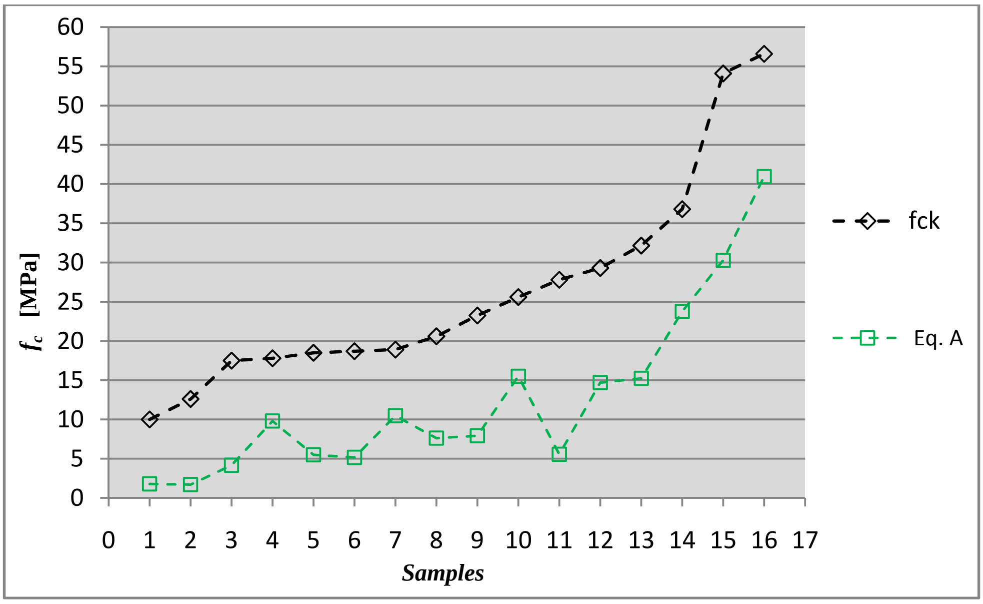


Fig. 4.2. Diagram "f_c – Test number" obtained for Bilgehan&Turgut's formulation

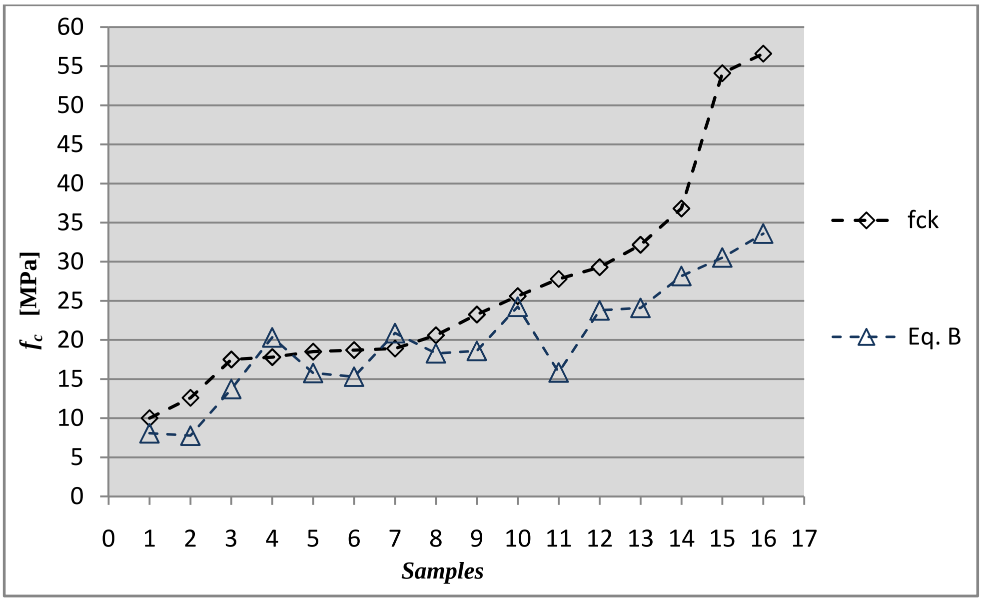


Fig. 4.3. Diagram "f_c – Test number" obtained for R. Giannini's formulation

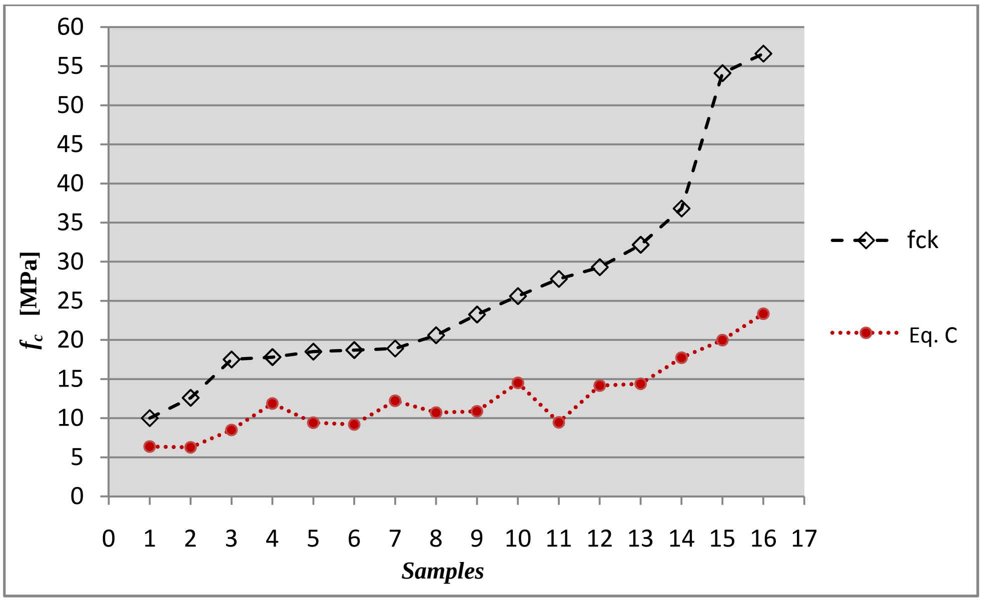


Fig. 4.4. Diagram "f_c – Test number" obtained for Y. Hisham-Qasrawi's formulation

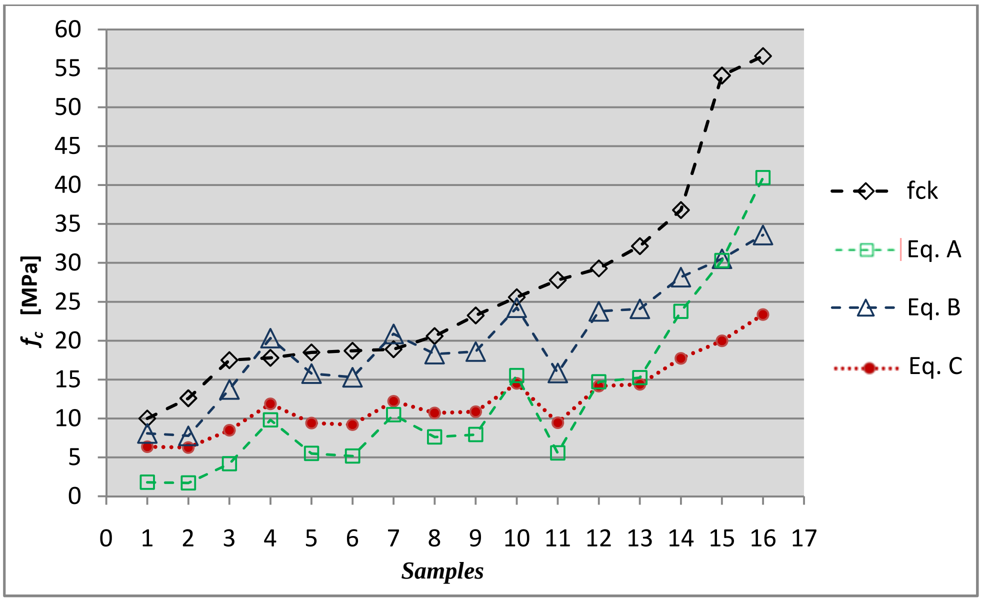


Fig. 4.5. Comparison between the different "f_c – Test number" diagrams.

4.2.2 Analysis with $ANN_{(UPV)}$

In order to understand the influence of the number of connections in a 3-layered feed-forward network with the only ultrasonic pulse velocity as input parameter, nine ANNs were developed. The increment steps between all the networks were established on the basis of the efficiency-computational time ratio.

N.	f_c [Mpa]	UPV [m/s]	*Net 1-5-1*	*Net 1-10-1*	*Net 1-15-1*	*Net 1-20-1*	*Net 1-25-1*	*Net 1-30-1*	*Net 1-50-1*	*Net 1-70-1*	*Net 1-90-1*
1	10,00	2470	11,2616	11,3506	11,2852	11,1276	11,1349	11,2950	11,2835	10,3817	10,3675
2	12,60	2450	10,9298	11,3118	11,3270	11,5771	11,5687	11,4001	11,4215	12,2423	12,2535
3	17,50	2830	19,0573	16,9982	17,0467	16,6820	16,6910	16,7664	16,7613	17,2976	17,3841
4	17,80	3250	20,3467	18,5981	18,2482	18,4319	18,3308	18,0555	18,0052	17,7763	18,1797
5	18,50	2960	21,3034	22,0748	22,0413	22,1645	22,1502	22,1249	22,1101	22,5928	22,7246
6	18,70	2930	20,9391	20,9490	20,9633	21,1544	21,1284	20,9582	20,9527	19,1564	18,8367
7	18,90	3285	20,6449	19,3254	19,0580	18,9603	18,9271	19,0289	18,9598	19,1610	18,7707
8	20,60	3120	21,0874	22,1140	22,2532	22,0398	22,1155	22,3622	22,4024	22,7426	22,4091
9	23,25	3140	20,8844	21,2954	21,4507	21,3878	21,4461	21,4866	21,5394	21,2020	21,3956
10	25,60	3500	28,3190	29,1178	28,6576	28,8260	28,6975	28,3719	28,1411	28,0893	28,1052
11	27,80	2965	21,3522	22,2461	22,2048	22,3076	22,2962	22,3024	22,2860	23,1698	23,3470
12	29,30	3470	26,8000	28,2024	29,2316	29,0909	29,2926	29,6688	29,9796	30,0324	30,1028
13	32,16	3490	27,8031	28,8416	28,9409	28,9811	28,9643	28,9185	28,8955	28,8925	28,8620
14	36,80	3750	42,1944	37,5234	37,1278	36,8967	36,8428	36,8760	36,8166	36,7690	36,7609
15	54,10	3900	49,7305	53,4169	53,5467	53,8719	53,9429	53,8965	53,9991	54,0959	54,1075
16	56,60	4095	57,5465	56,8516	56,8320	56,7123	56,6820	56,7004	56,6574	56,6072	56,6016
RMSE [Mpa]			3,1457	2,3048	2,2196	2,2246	2,2107	2,1909	2,1781	2,0152	1,9786

Tab. 4.2. Summary of $ANN_{(UPV)}$ data and correlations for on-site concrete strength

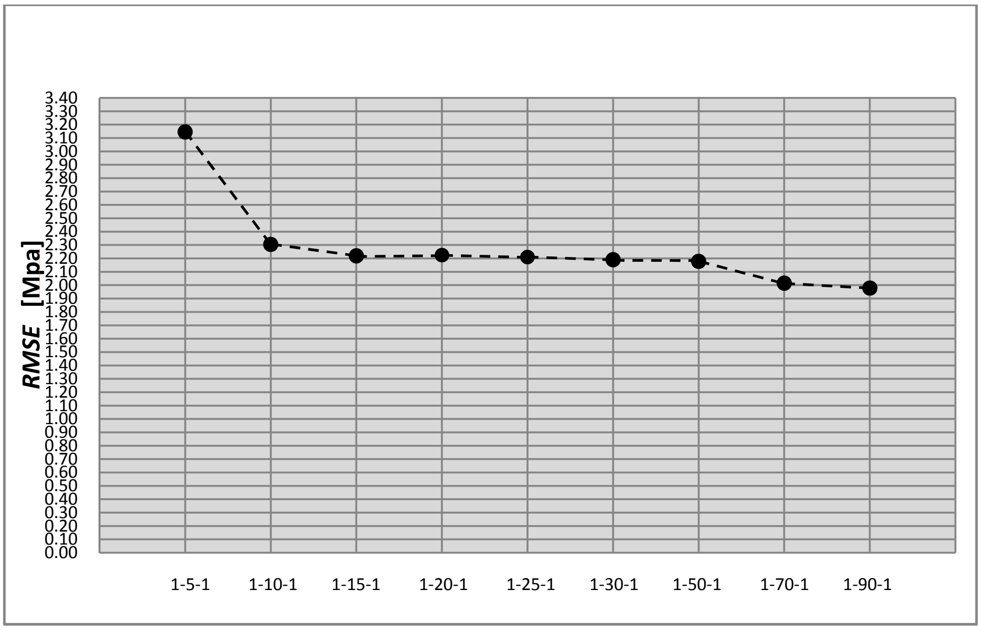


Fig. 4.6. Comparison between the different ANNs performance.

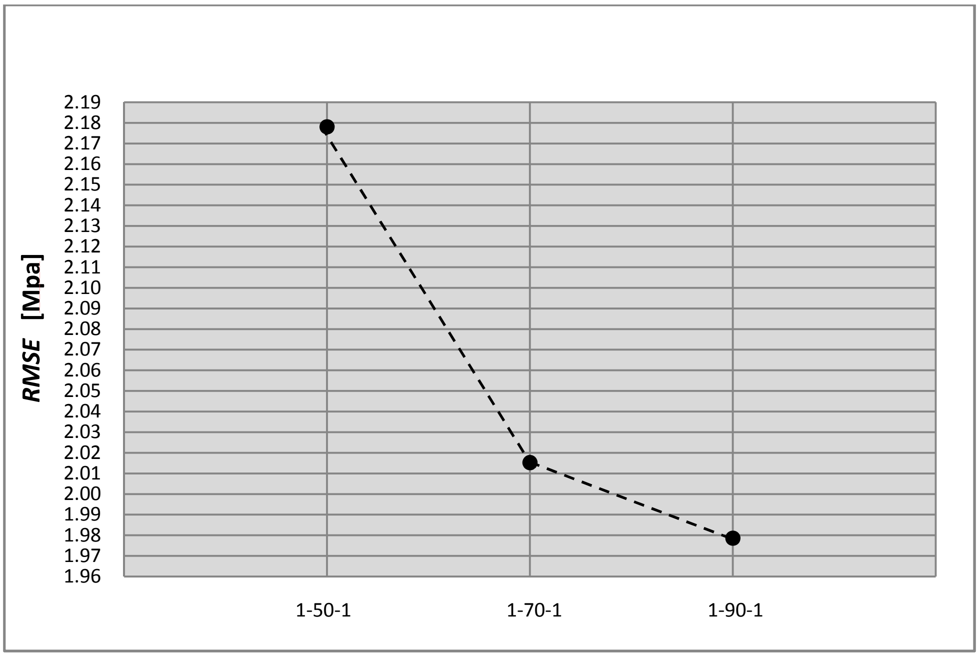


Fig. 4.7. Comparison between the different ANNs performance: particular of the last three ANNs architecture with the most reliability in concrete strength assessment.

ANN$_{(UPV)}$ 1-5-1

In the following diagrams were represented the training and learning phases of the ANN$_{(UPV)}$ with 5 neurons in the hidden layer. The principal data of this network is reported in *Tab. 4.3.*

Net Train Param. Epochs	***Best Performance Epochs***	**RMSE [MPa]**
1000	438	3,1457

Tab. 4.3. Summary of 1-5-1 ANN$_{(UPV)}$ data.

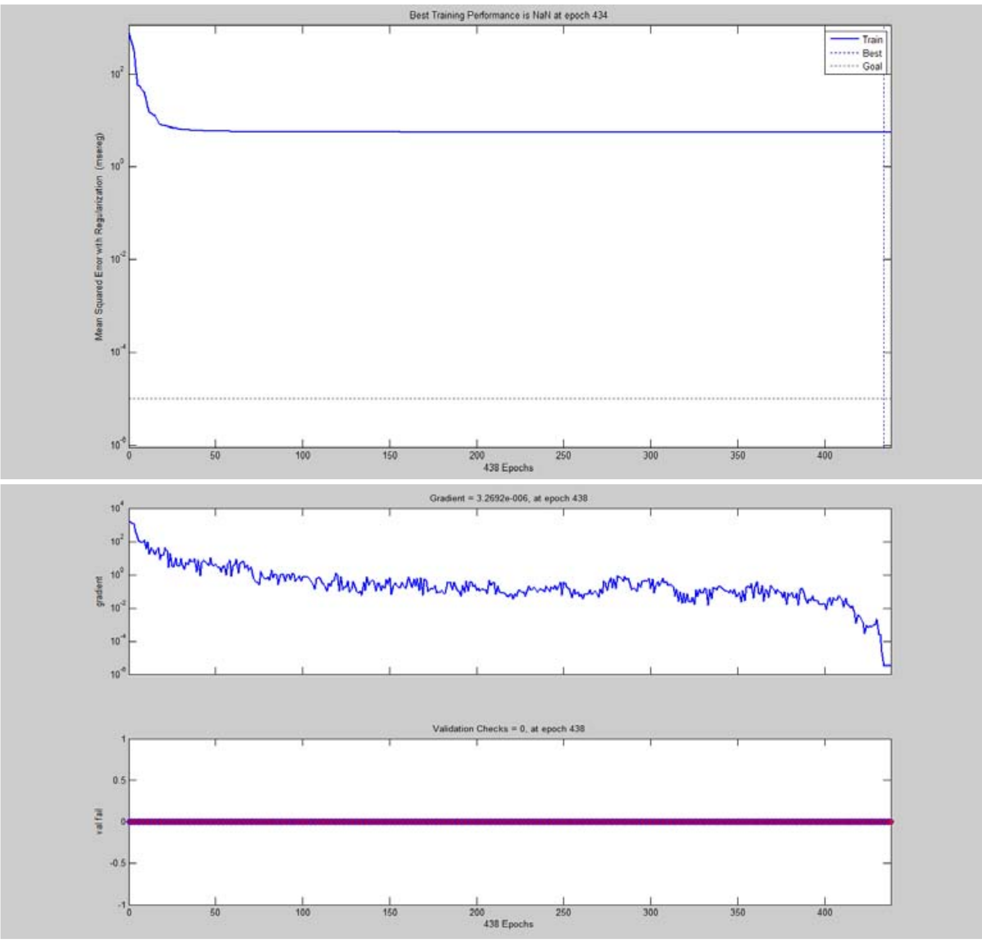


Fig. 4.8. ANN behavior during training performance and gradient descendent.

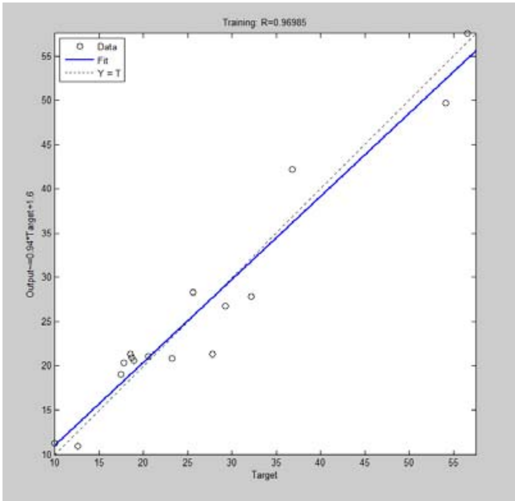


Fig. 4.9. ANN training set.

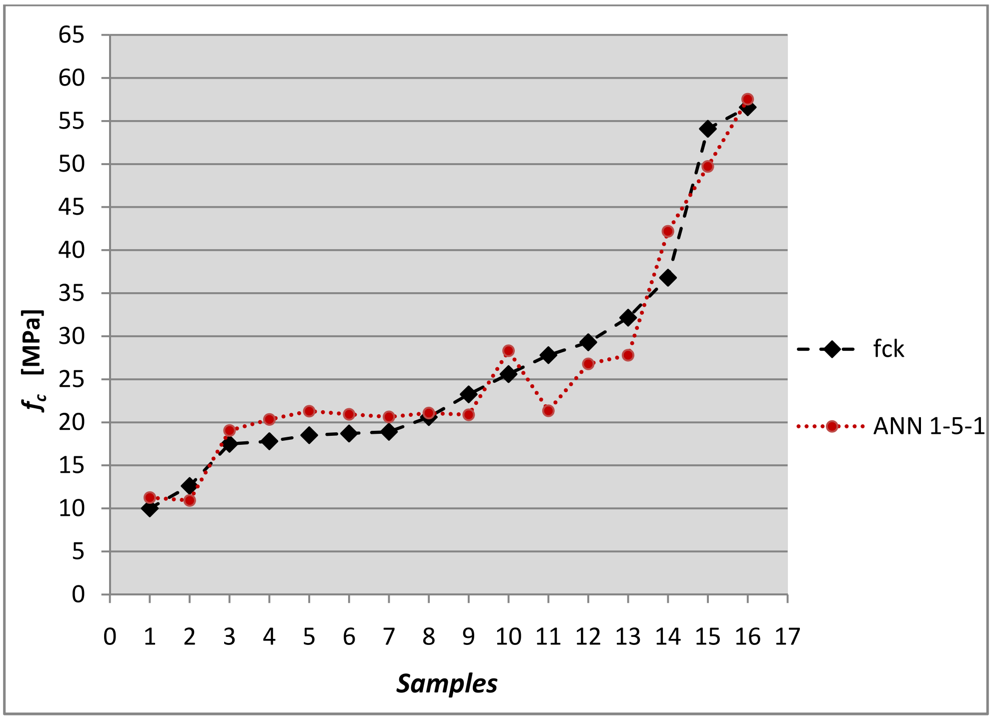


Fig. 4.10. Diagram "f_c – Test number" obtained for $ANN_{(UPV)}$ 1-5-1

ANN$_{(UPV)}$ 1-10-1

In the following diagrams were represented the training and learning phases of the ANN$_{(UPV)}$ with 10 neurons in the hidden layer. The principal data of this network is reported in *Tab. 4.4.*

Net Train Param. Epochs	***Best Performance Epochs***	**RMSE [MPa]**
2000	1384	2,3048

Tab. 4.4. Summary of 1-10-1 ANN$_{(UPV)}$ data.

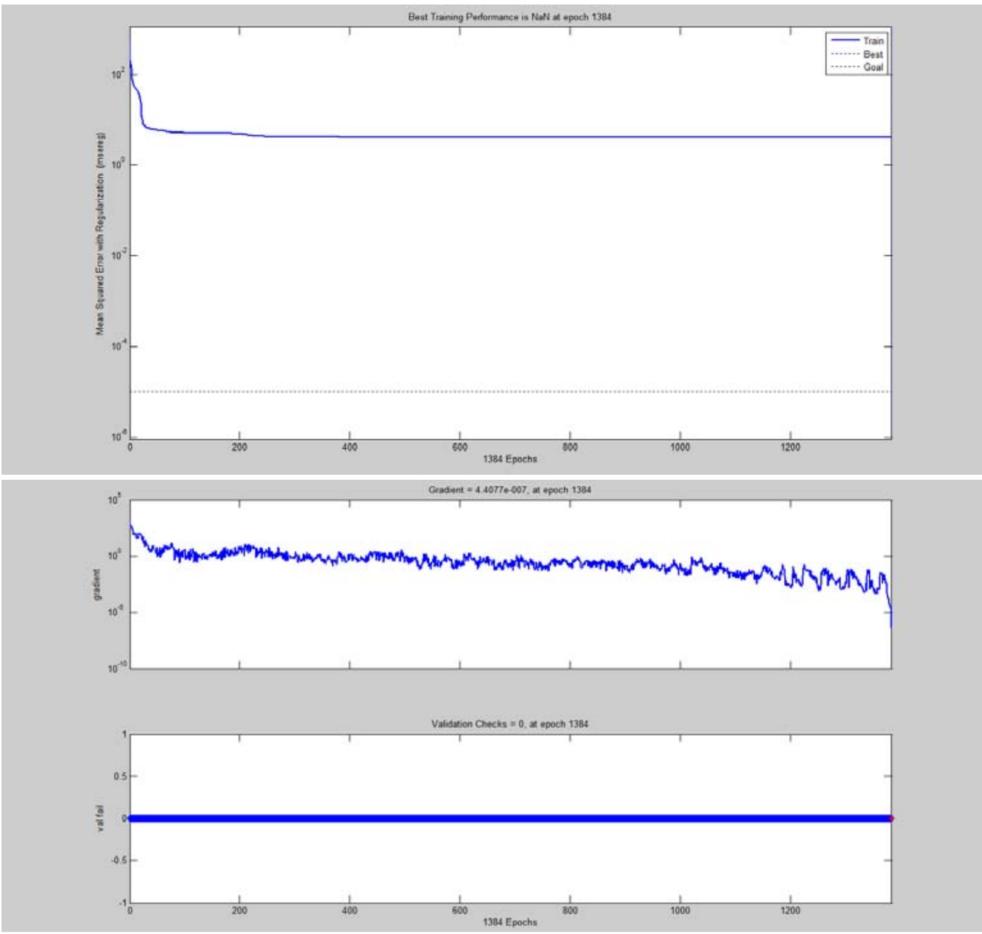


Fig. 4.11. ANN behavior during training performance and gradient descendent.

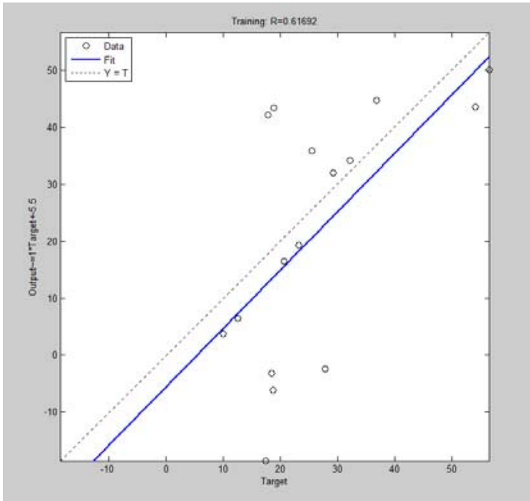


Fig. 4.12. ANN training set.

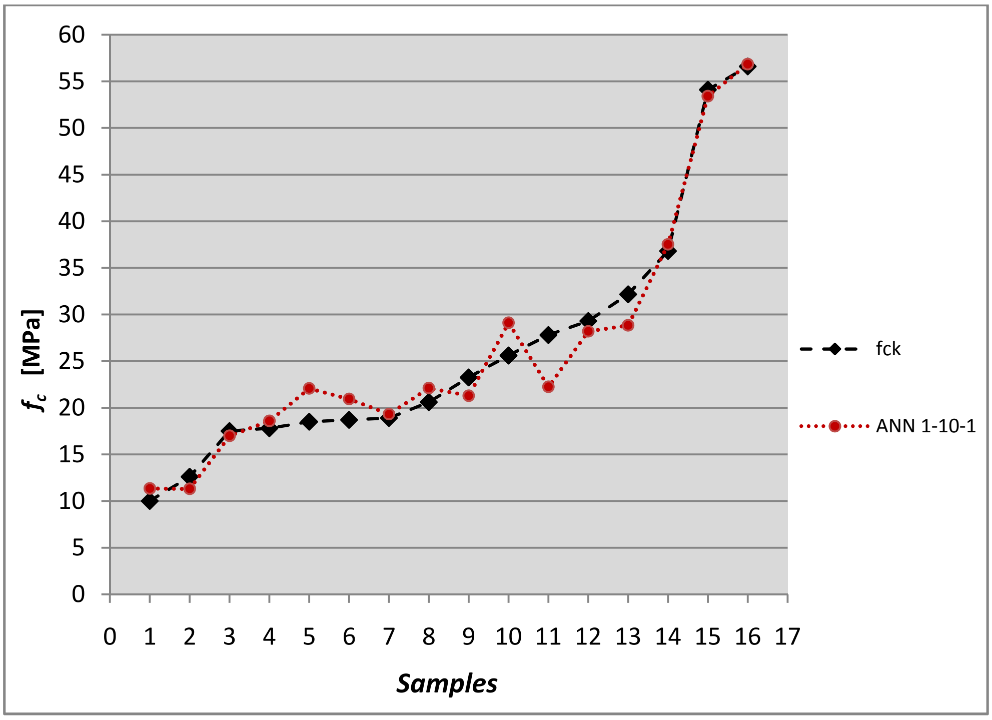


Fig. 4.13. Diagram “f_c – Test number” obtained for $ANN_{(UPV)}$ 1-10-1

ANN$_{(UPV)}$ 1-15-1

In the following diagrams were represented the training and learning phases of the ANN$_{(UPV)}$ with 15 neurons in the hidden layer. The principal data of this network is reported in *Tab. 4.5.*

Net Train Param. Epochs	***Best Performance Epochs***	**RMSE [MPa]**
3000	1425	2,2196

Tab. 4.5. Summary of 1-15-1 ANN$_{(UPV)}$ data.

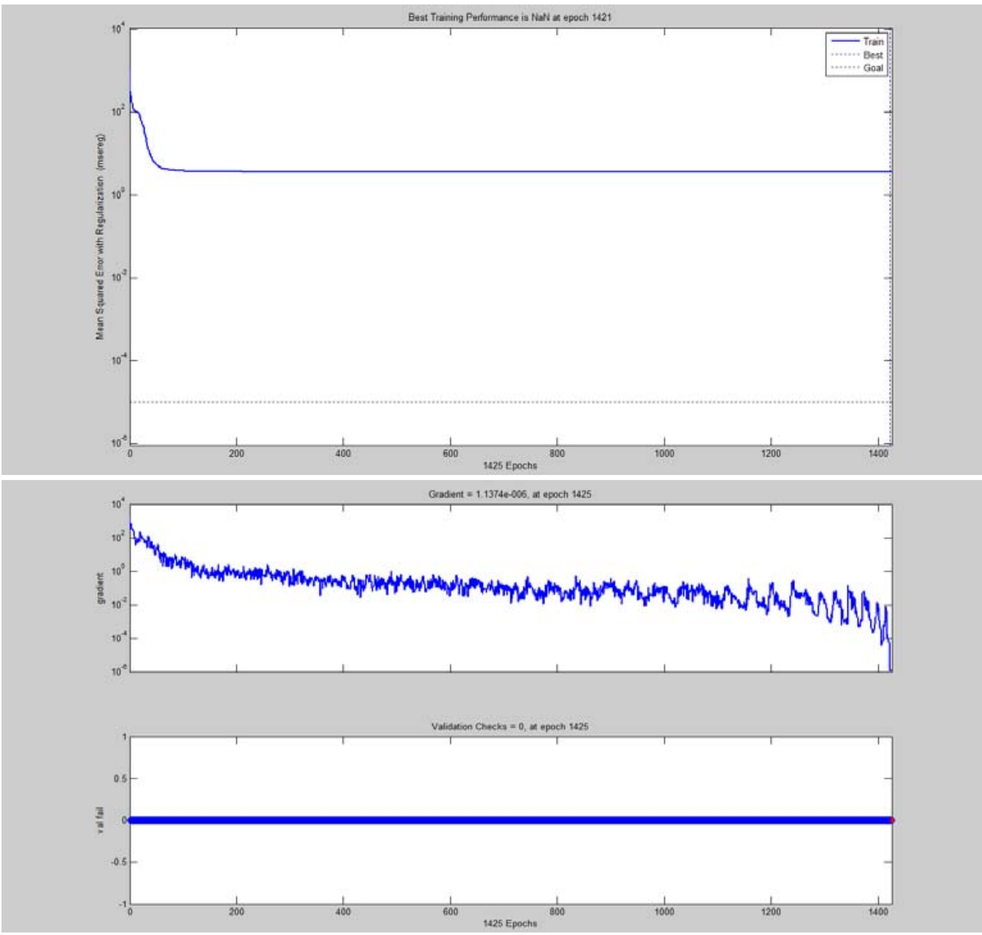


Fig. 4.14. ANN behavior during training performance and gradient descendent.

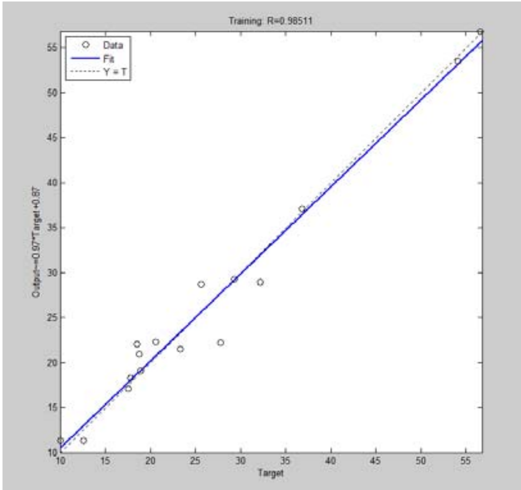


Fig. 4.15. ANN training set.

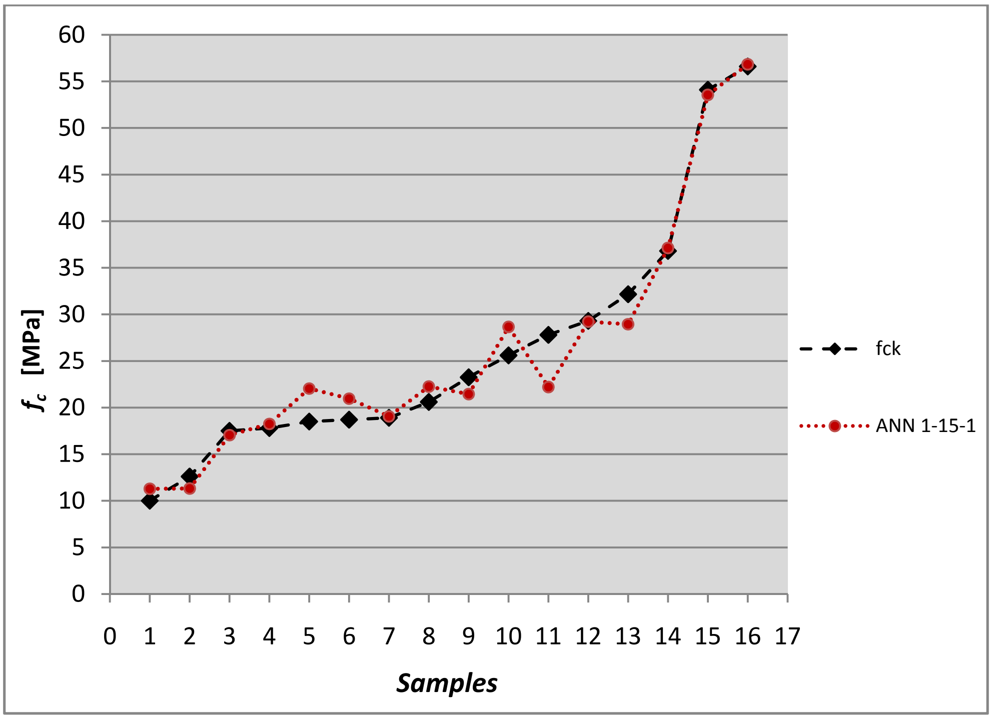


Fig. 4.16. Diagram "f_c – Test number" obtained for $ANN_{(UPV)}$ 1-15-1

ANN$_{(UPV)}$ 1-20-1

In the following diagrams were represented the training and learning phases of the ANN$_{(UPV)}$ with 20 neurons in the hidden layer. The principal data of this network is reported in *Tab. 4.6.*

Net Train Param. Epochs	***Best Performance Epochs***	**RMSE [MPa]**
5000	2337	2,2246

Tab. 4.6. Summary of 1-20-1 ANN$_{(UPV)}$ data.

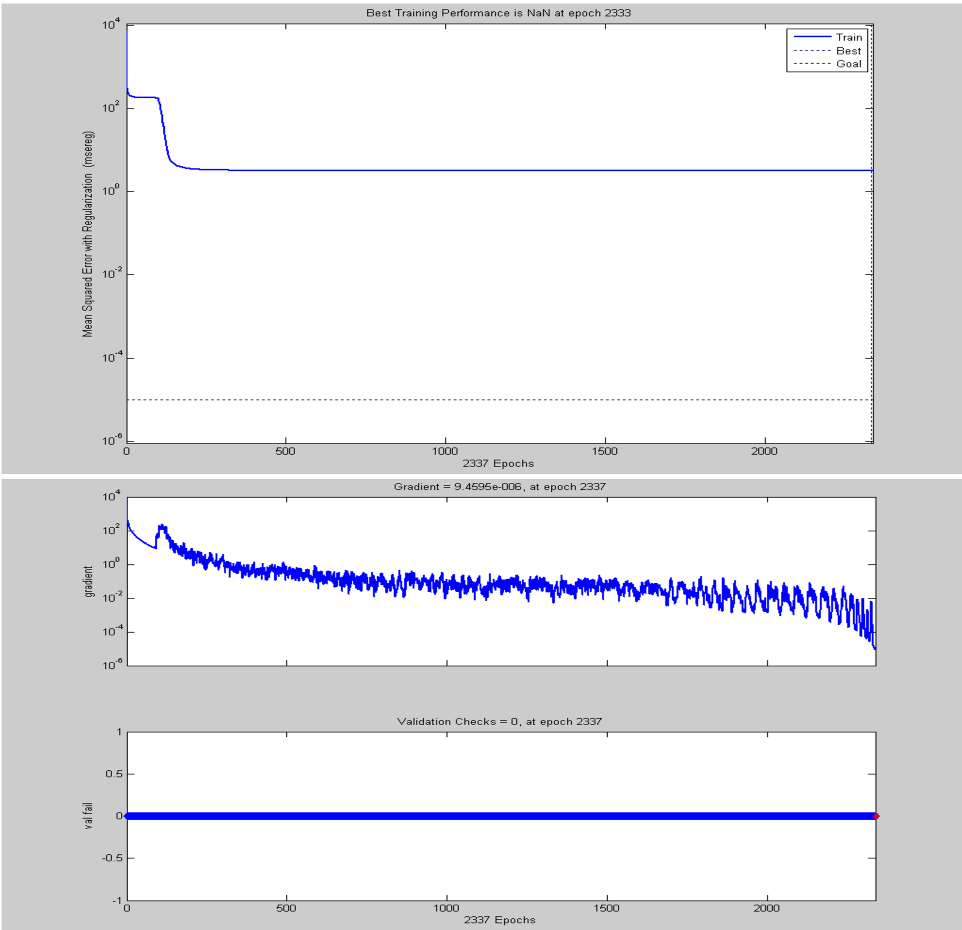


Fig. 4.17. ANN behavior during training performance and gradient descendent.

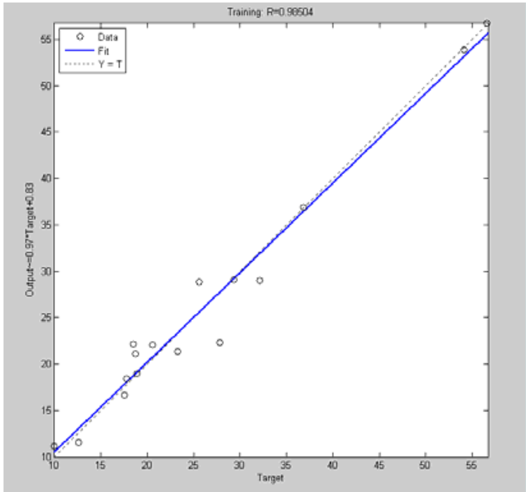


Fig. 4.18. ANN training set.

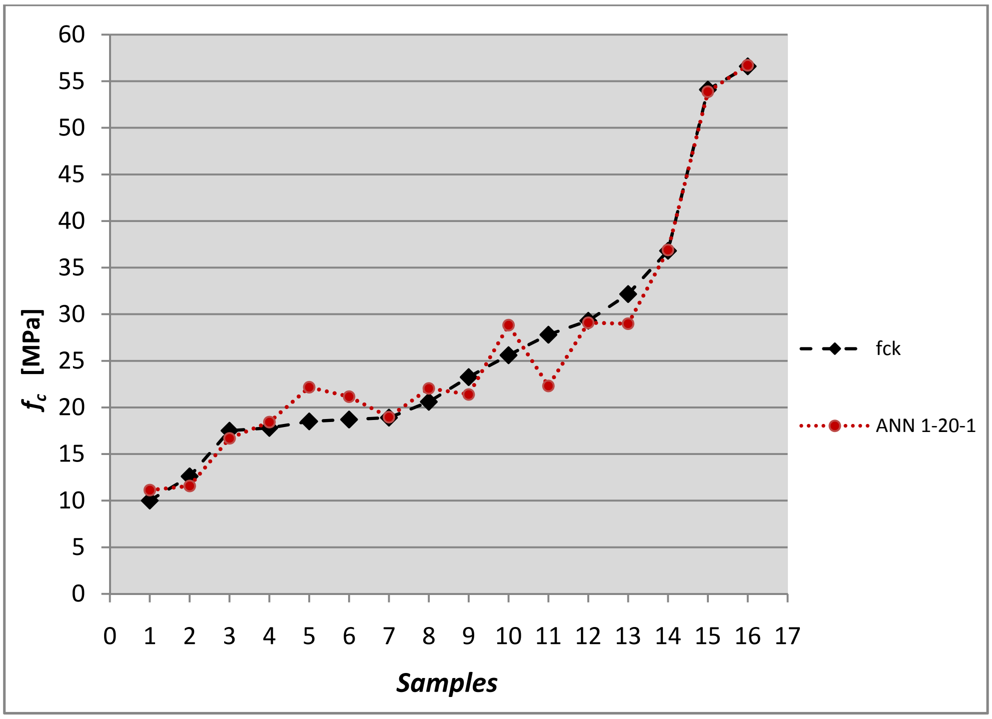


Fig. 4.19. Diagram "f_c – Test number" obtained for $ANN_{(UPV)}$ 1-20-1

ANN$_{(UPV)}$ 1-25-1

In the following diagrams were represented the training and learning phases of the ANN$_{(UPV)}$ with 25 neurons in the hidden layer. The principal data of this network is reported in *Tab. 4.7.*

Net Train Param. Epochs	***Best Performance Epochs***	**RMSE [MPa]**
7000	4965	2,2107

Tab. 4.7. Summary of 1-25-1 ANN$_{(UPV)}$ data.

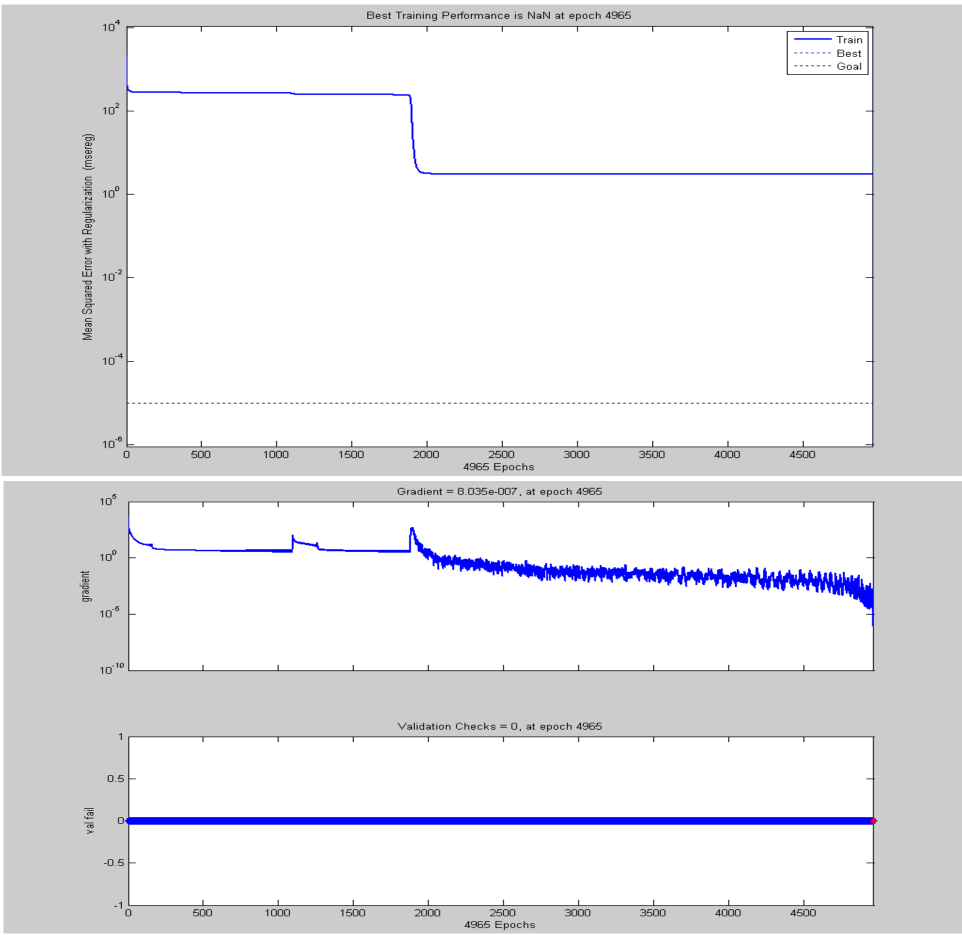


Fig. 4.20. ANN behavior during training performance and gradient descendent.

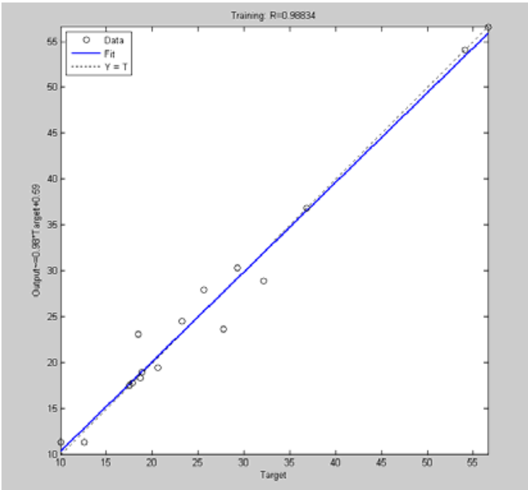


Fig. 4.21. ANN training set.

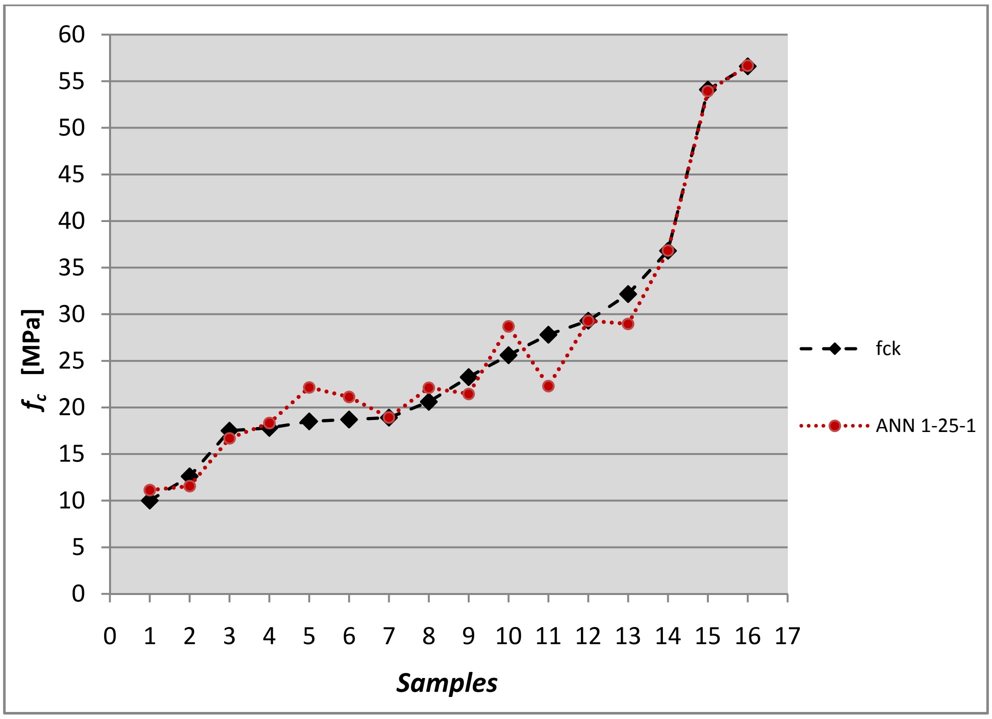


Fig. 4.22. Diagram "f_c – Test number" obtained for ANN$_{(UPV)}$ 1-25-1

ANN$_{(UPV)}$ 1-30-1

In the following diagrams were represented the training and learning phases of the ANN$_{(UPV)}$ with 30 neurons in the hidden layer. The principal data of this network is reported in *Tab. 4.8.*

Net Train Param. Epochs	***Best Performance Epochs***	**RMSE [MPa]**
10000	5124	2,1909

Tab. 4.8. Summary of 1-30-1 ANN$_{(UPV)}$ data.

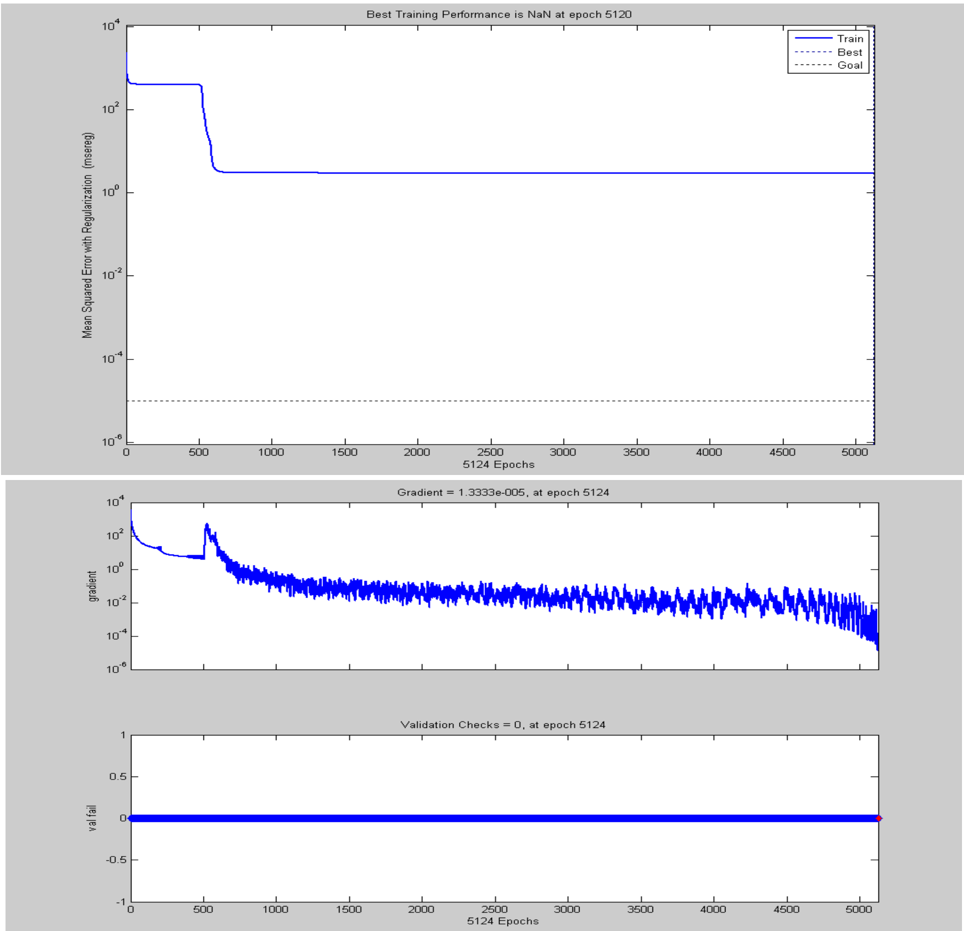


Fig. 4.23. ANN behavior during training performance and gradient descendent.

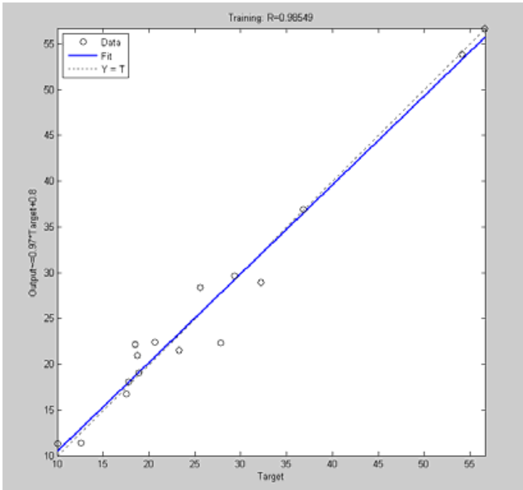


Fig. 4.24. ANN training set.

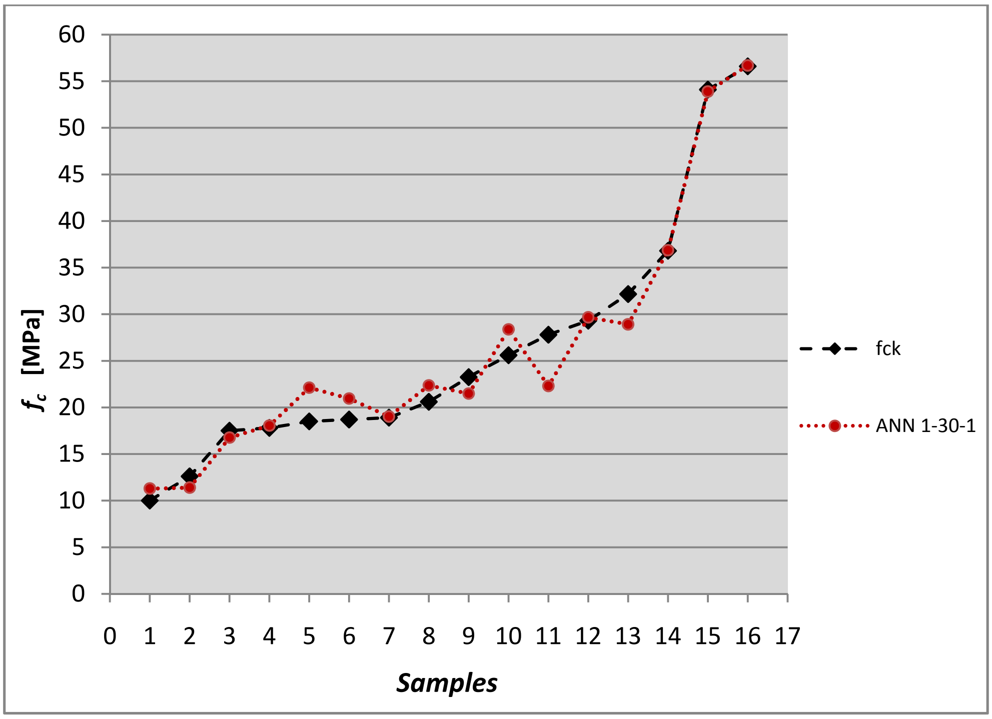


Fig. 4.25. Diagram "f_c – Test number" obtained for $ANN_{(UPV)}$ 1-30-1

ANN$_{(UPV)}$ 1-50-1

In the following diagrams were represented the training and learning phases of the ANN$_{(UPV)}$ with 50 neurons in the hidden layer. The principal data of this network is reported in *Tab. 4.9.*

Net Train Param. *Epochs*	*Best Performance* *Epochs*	**RMSE** **[MPa]**
20000	7298	2,1781

Tab. 4.9. Summary of 1-50-1 ANN$_{(UPV)}$ data.

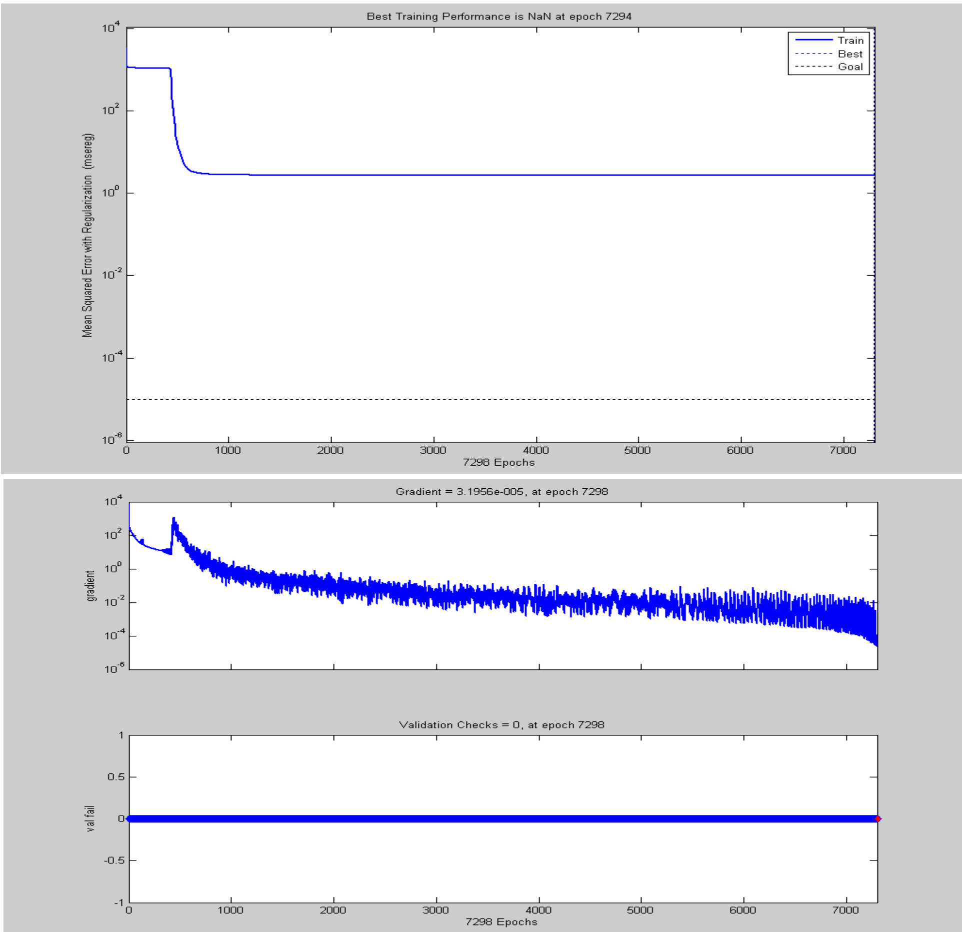


Fig. 4.26. ANN behavior during training performance and gradient descendent.

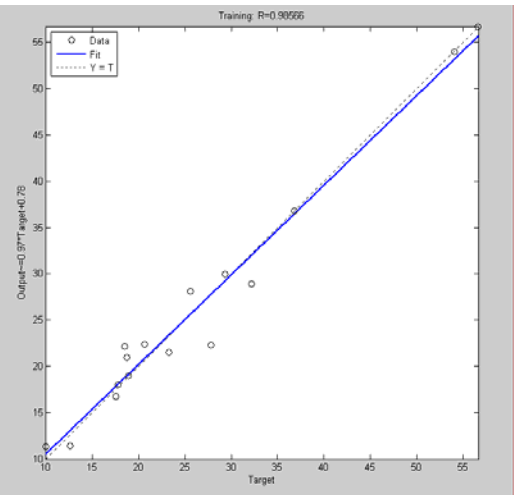


Fig. 4.27. ANN training set.

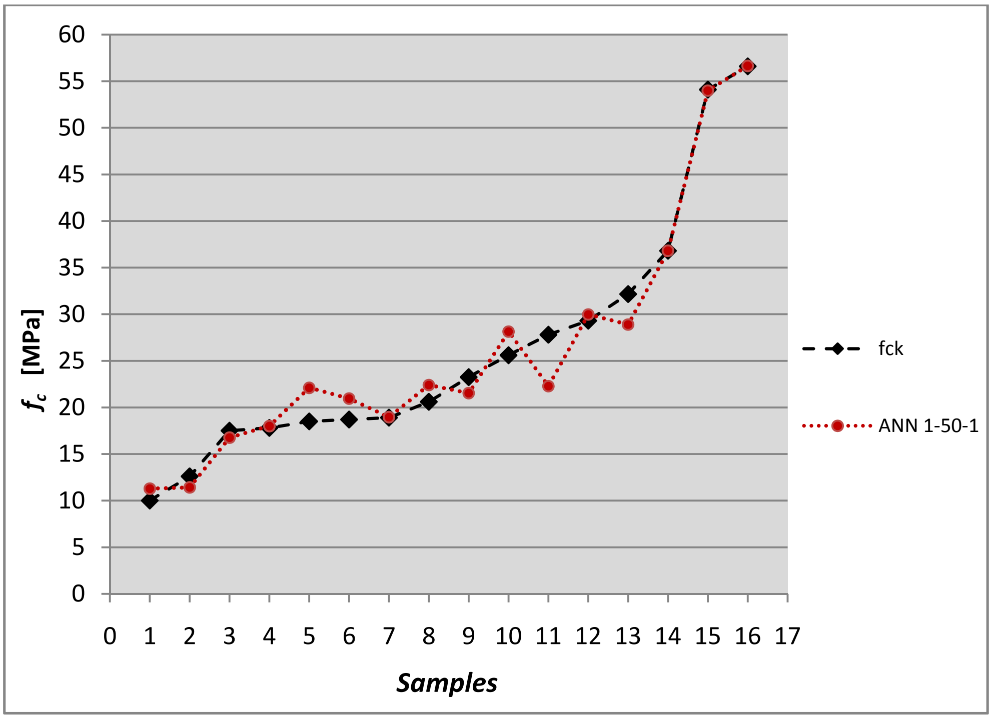


Fig. 4.28. Diagram "f_c – Test number" obtained for $ANN_{(UPV)}$ 1-50-1

ANN$_{(UPV)}$ 1-70-1

In the following diagrams were represented the training and learning phases of the ANN$_{(UPV)}$ with 70 neurons in the hidden layer. The principal data of this network is reported in *Tab. 4.10.*

Net Train Param. Epochs	***Best Performance Epochs***	**RMSE [MPa]**
40000	17603	2,0152

Tab. 4.10. Summary of 1-70-1 ANN$_{(UPV)}$ data.

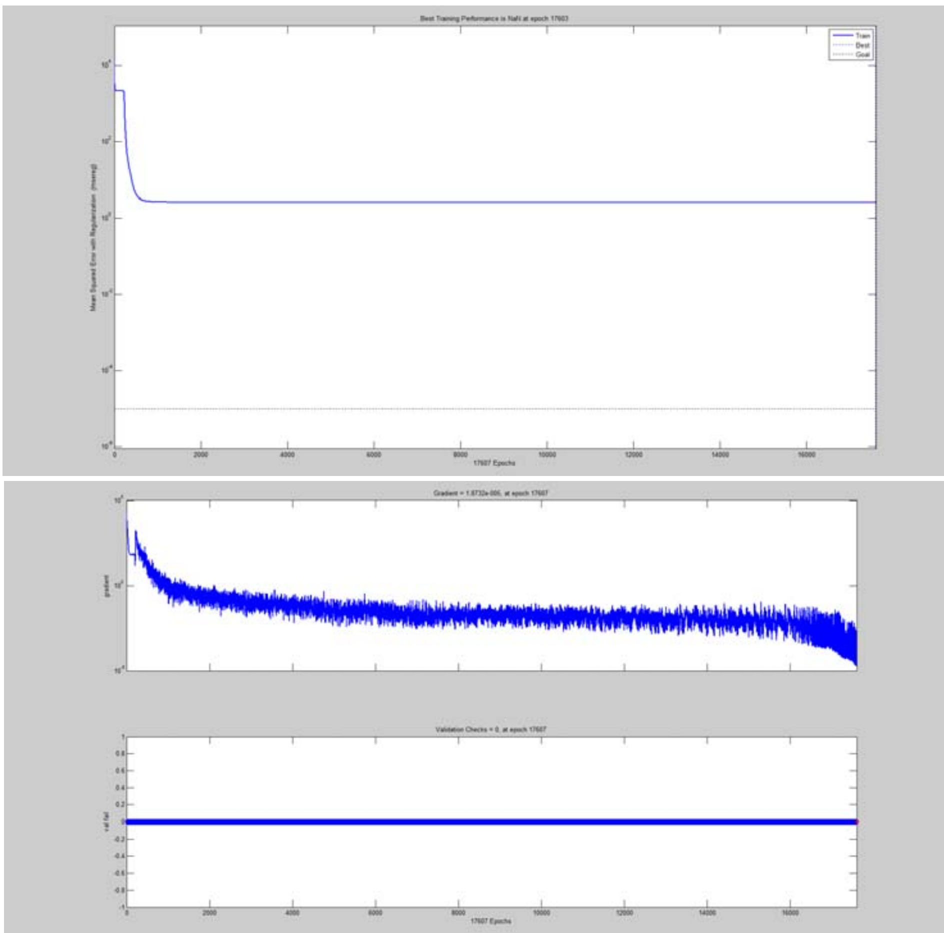


Fig. 4.29. ANN behavior during training performance and gradient descendent.

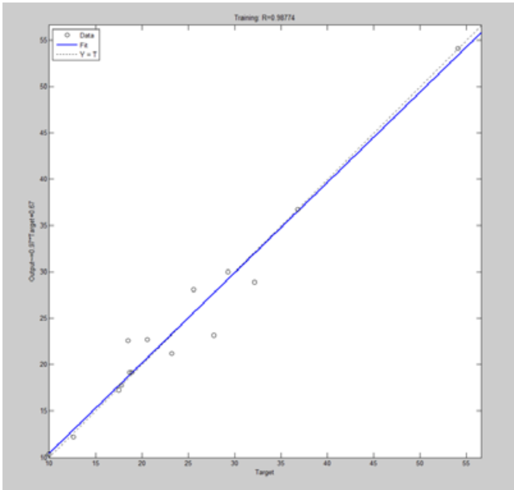


Fig. 4.30. ANN training set.

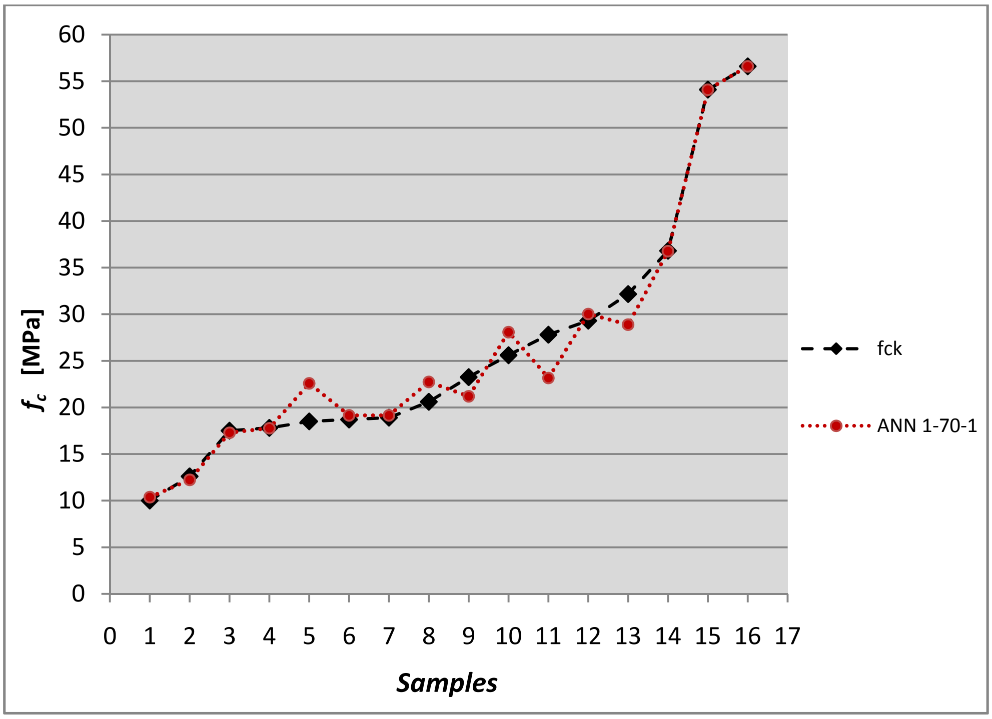


Fig. 4.31. Diagram "f_c – Test number" obtained for $ANN_{(UPV)}$ 1-70-1

ANN$_{(UPV)}$ 1-90-1

In the following diagrams were represented the training and learning phases of the ANN$_{(UPV)}$ with 90 neurons in the hidden layer. The principal data of this network is reported in *Tab. 4.11*.

Net Train Param. Epochs	***Best Performance Epochs***	**RMSE [MPa]**
100000	21943	1,9786

Tab. 4.11. Summary of 1-90-1 ANN$_{(UPV)}$ data.

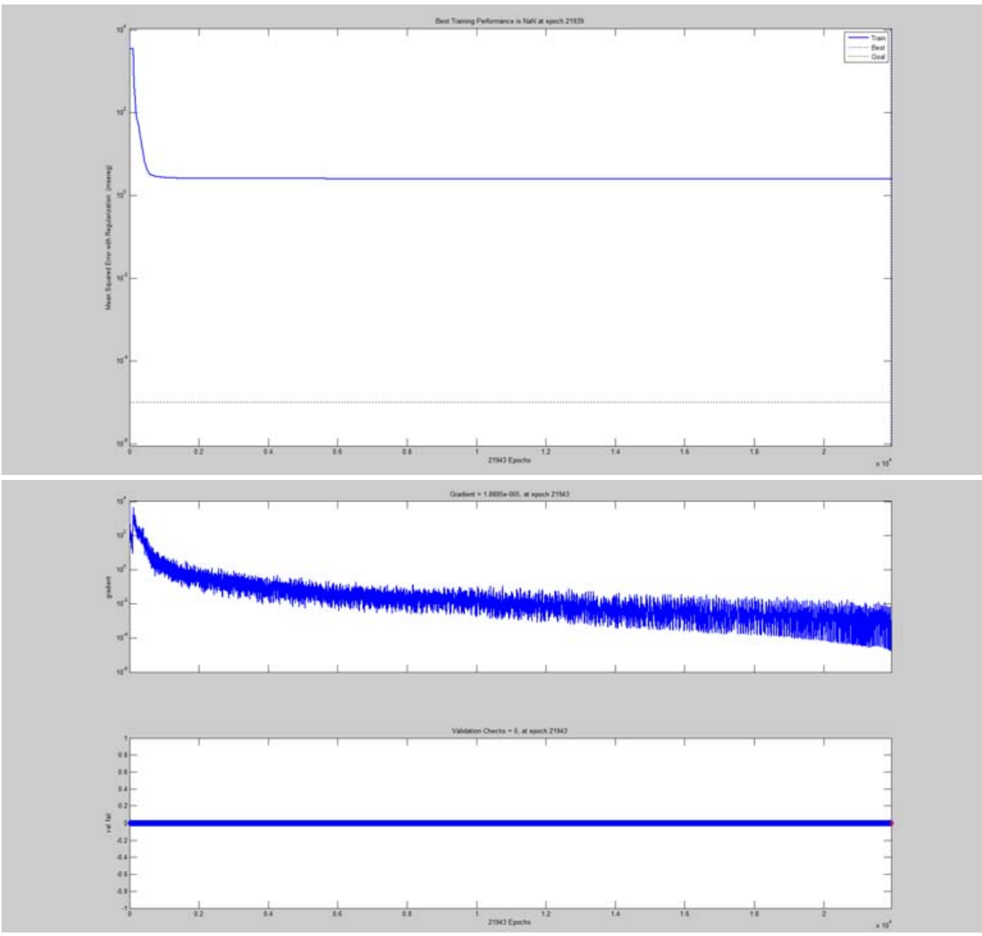


Fig. 4.32. ANN behavior during training performance and gradient descendent.

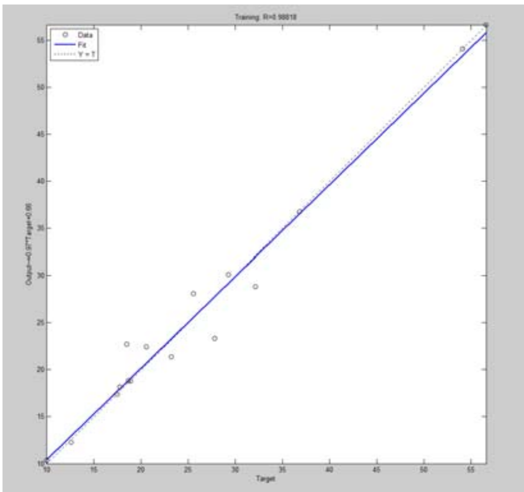


Fig. 4.33. ANN training set.

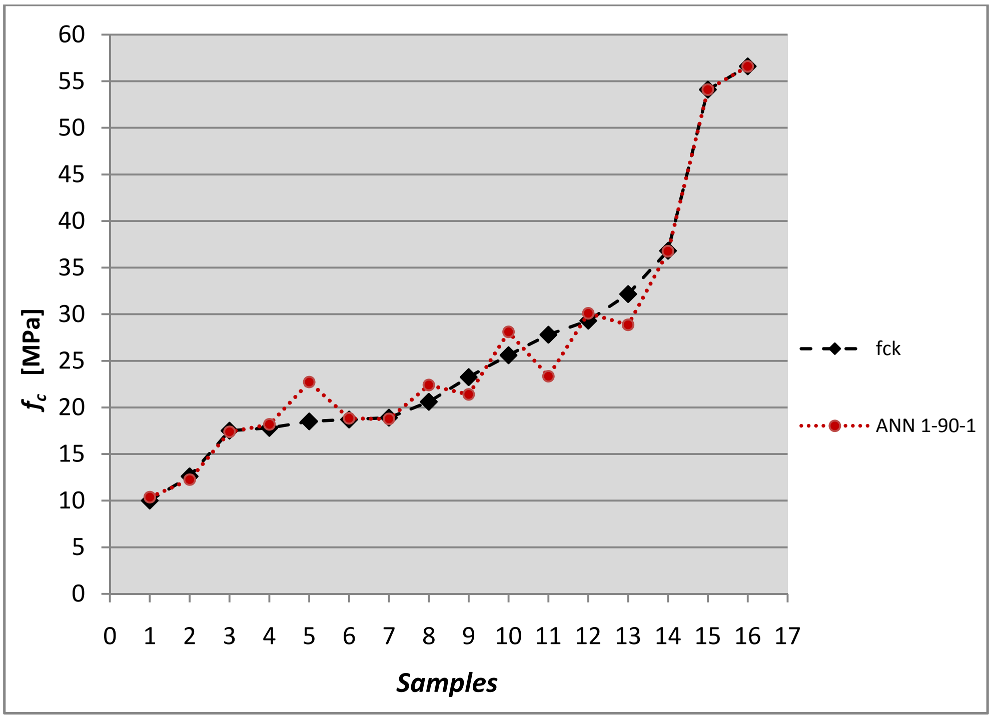


Fig. 4.34. Diagram "f_c – Test number" obtained for ANN$_{(UPV)}$ 1-90-1

4.3 Comparison between SonReb methods – $ANN_{(SonReb)}$

4.3.1 Analysis with SonReb combined method

Assessments made with the regression formulas found in the literature (see *Chapter 2.5.3.*) and suggested by Technical Standard of Toscana Region have showed substantially the similar behavior in terms of efficiency. The most effective formulation evaluated in terms of RMSE was that of *J. Gasparik*, 1992 (*Eq. C*), as it can be seen as follows.

N.	f_c [Mpa]	RI	UPV [m/s]	$f_{c\,(A)}$ [Mpa]	$f_{c\,(B)}$ [Mpa]	$f_{c\,(C)}$ [Mpa]	$f_{c\,(D)}$ [Mpa]
1	10,00	34,72	2470	7,24	10,08	12,57	9,96
2	12,60	38,89	2450	8,39	11,25	14,36	11,33
3	17,50	36,39	2830	11,12	14,92	17,26	14,43
4	17,80	31,90	3250	13,26	18,21	18,93	16,80
5	18,50	37,22	2960	12,90	17,06	19,30	16,42
6	18,70	38,83	2930	13,34	17,40	19,96	16,90
7	18,90	34,61	3285	15,28	20,38	21,37	19,01
8	20,60	39,34	3120	15,98	20,57	22,78	19,78
9	23,25	39,78	3140	16,51	21,14	23,38	20,34
10	25,60	36,84	3500	19,66	25,42	25,97	23,68
11	27,80	39,00	2965	13,83	17,99	20,52	17,45
12	29,30	41,44	3470	22,67	28,19	29,60	26,82
13	32,16	38,39	3490	20,68	26,37	27,20	24,75
14	36,80	45,45	3750	31,57	37,58	38,34	35,83
15	54,10	47,28	3900	36,94	43,12	43,23	41,10
16	56,60	47,33	4095	42,01	48,65	47,46	46,04
RMSE [Mpa]				8,5422	4,5720	4,3734	5,4891

Tab. 4.12. Summary of SonReb data and correlations for on-site concrete strength

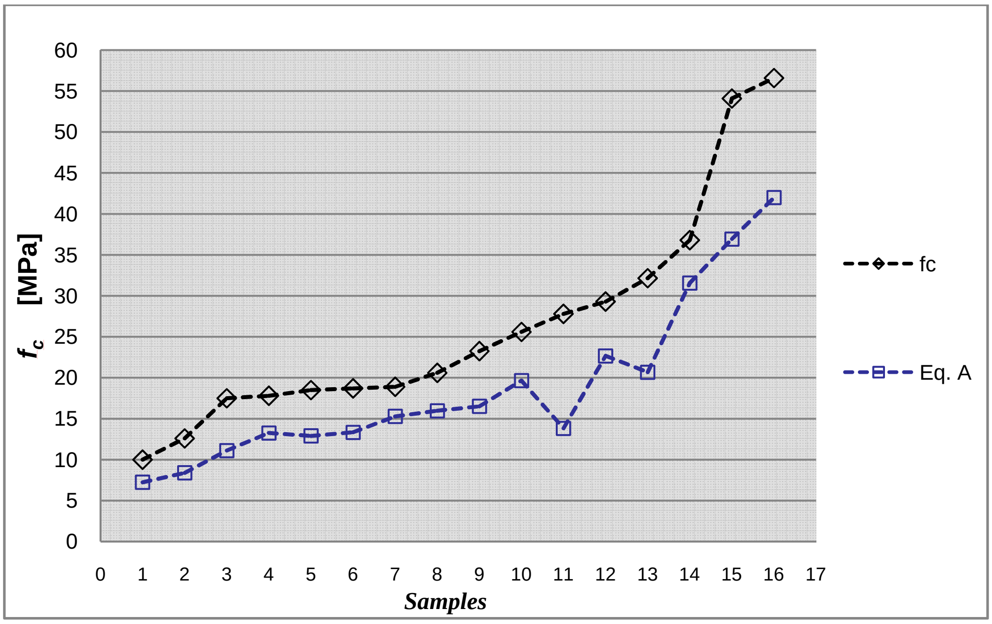


Fig. 4.35. Diagram "f_c – Test number" obtained for Giacchetti-Lacquaniti's formulation

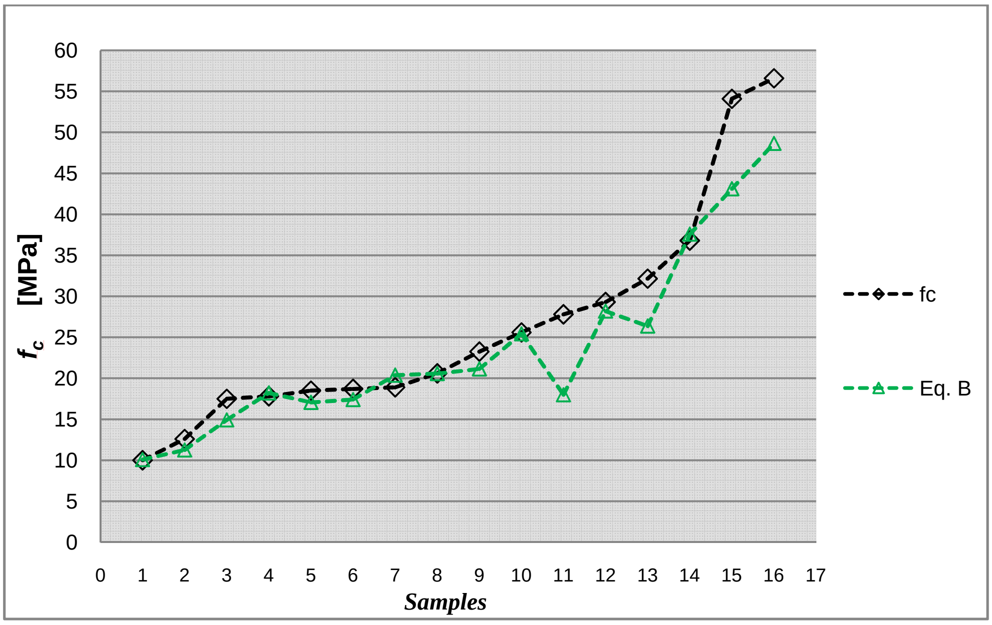


Fig. 4.36. Diagram "f_c – Test number" obtained for Di Leo-Pascale's formulation

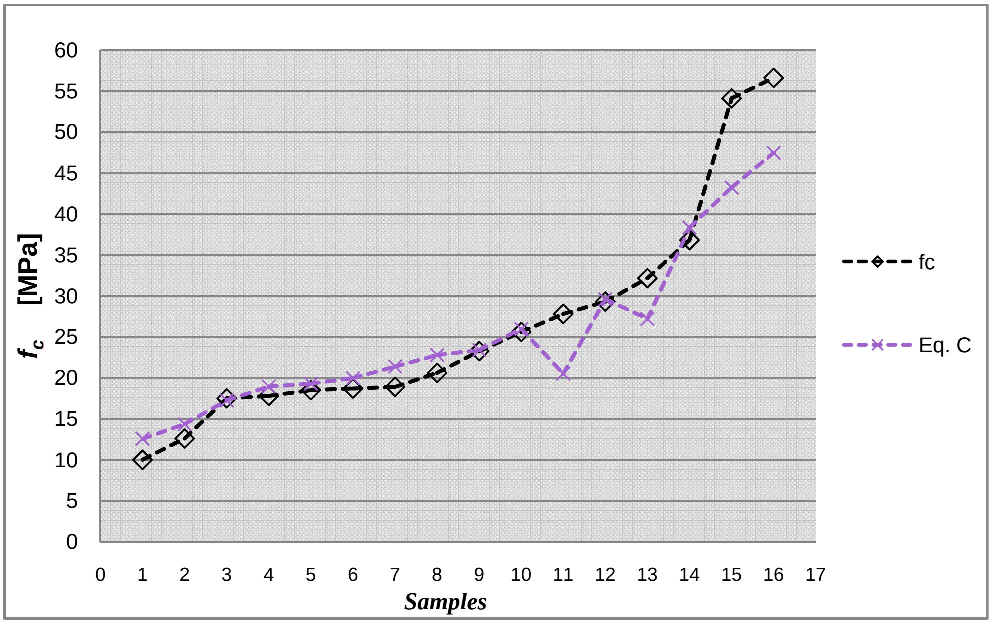


Fig. 4.37. Diagram "f_c – Test number" obtained for Gasparik's formulation

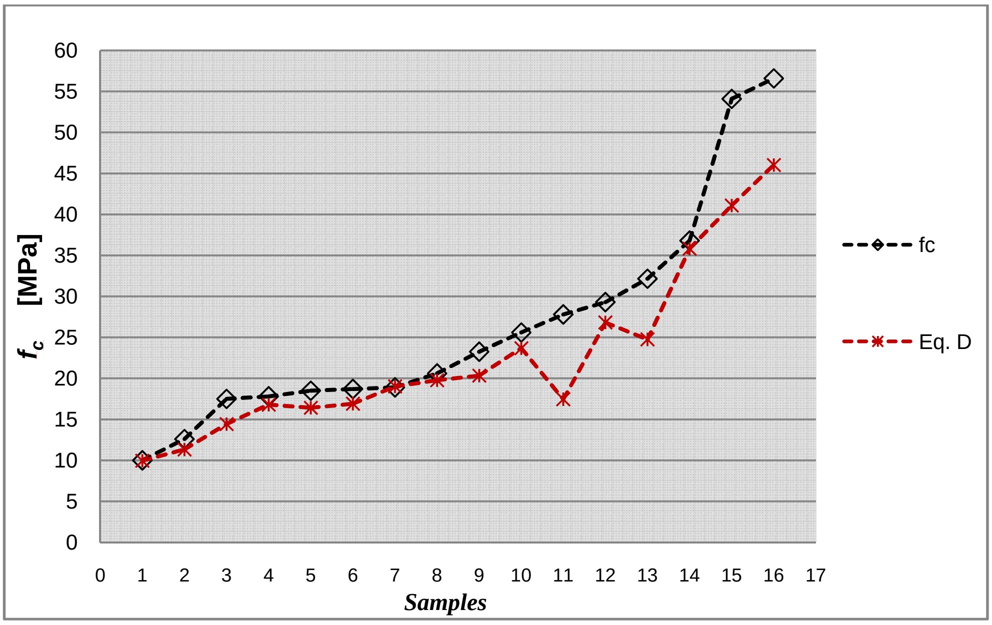


Fig. 4.38. Diagram "f_c – Test number" obtained for the mean values of the previous formulation, as suggested by Technical Standards of Toscana Region.

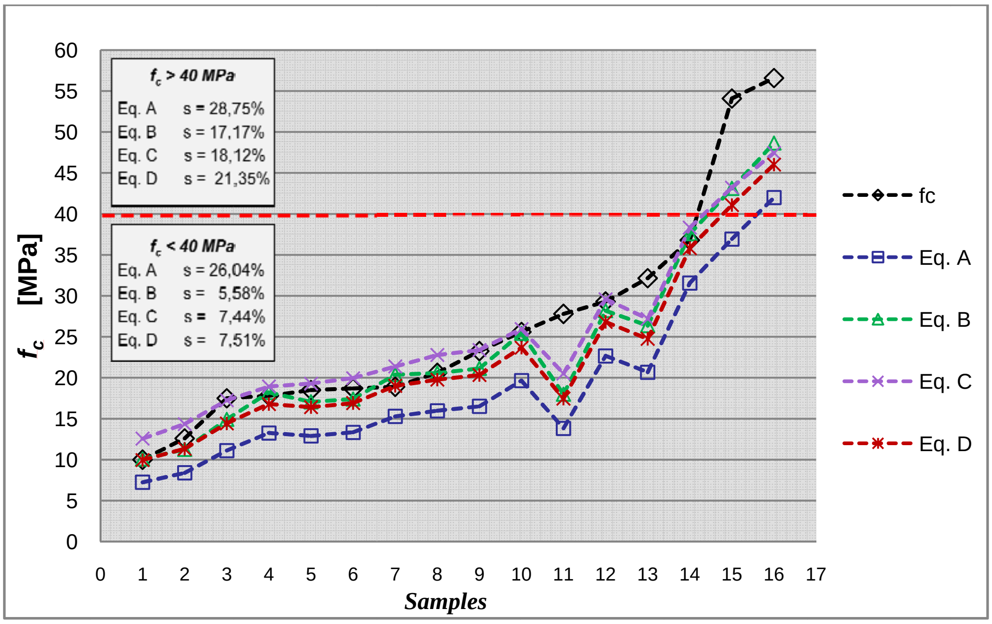


Fig. 4.39. Comparison between the different formulations.

With regards to the SonReb combined method all the expressions taken into account have shown a progress similar to the concrete strength curve (*Fig. 4.39.*). Excluding the Eq. A that presents values too much far from the actual onsite, the other three expressions are very close to the referenced f_c curve. In particular for the Eq. B and Eq. C it can be observed that the percentage of the root mean squared error for strength values superior than 40MPa is significant lower than the others two formulations, i.e. respectively 17,17% and 18,12% while for Eq. A is 28,75% and for Eq. D is 21,35%. This value strongly decreases if the range of data considered is minor than 40MPa. Finally if we consider as *SonReb curve* the mean values of the first three formulations (as Technical Standards of Toscana Region suggests to take into account), it can be seen that each data lay close to the f_c curve in a safety condition.

4.3.2 Analysis with $ANN_{(SonReb)}$

In order to understand the influence of the number of connections in a 3-layered feed-forward network with the two non-destructive parameters of rebound index and ultrasonic pulse velocity as input, nine ANNs were developed. The increment steps between all the networks were established on the basis of the efficiency-computational time ratio.

N.	f_c [Mpa]	RI	UPV [m/s]	*Net 2-5-1*	*Net 2-10-1*	*Net 2-15-1*	*Net 2-20-1*	*Net 2-25-1*	*Net 2-30-1*	*Net 2-50-1*	*Net 2-70-1*	*Net 2-90-1*
1	10,00	34,72	2470	12,4045	12,5107	12,4807	12,3810	12,4806	10,1252	10,0000	10,0000	10,0003
2	12,60	38,89	2450	14,1812	15,7506	13,8110	12,4210	13,8110	13,6588	12,6000	12,6001	12,5998
3	17,50	36,39	2830	16,2242	16,6681	17,3358	16,1923	16,3358	17,4761	17,5000	17,4999	17,4992
4	17,80	31,90	3250	17,5759	17,8196	17,8132	17,8548	17,8116	17,8220	17,8000	17,7999	17,7999
5	18,50	37,22	2960	18,3682	18,4567	18,4685	18,4242	18,4769	18,4369	18,4998	18,4998	18,4989
6	18,70	38,83	2930	22,4640	19,5950	19,2881	19,1786	19,0493	19,0621	18,6998	18,6999	18,7034
7	18,90	34,61	3285	19,0821	18,8114	18,8407	18,7993	18,8590	18,8803	18,9001	18,8999	18,8991
8	20,60	39,34	3120	21,8318	21,1724	20,9882	21,1203	20,8386	20,9991	20,5999	20,6001	20,6003
9	23,25	39,78	3140	23,9773	22,9694	23,0516	22,8894	23,1242	22,9554	23,2501	23,2502	23,2474
10	25,60	36,84	3500	26,0369	25,6881	25,6579	25,6741	25,6392	25,5776	25,5998	25,5999	25,5985
11	27,80	39,00	2965	22,6131	26,7146	27,0905	27,2633	27,3822	27,3983	27,7998	27,7998	27,7933
12	29,30	41,44	3470	28,8481	29,2246	29,2521	29,3104	29,2810	29,3188	29,2999	29,3000	29,2980
13	32,16	38,39	3490	31,5879	32,0823	32,1095	32,0834	32,1253	32,1612	32,1602	32,1598	32,1582
14	36,80	45,45	3750	37,3258	37,0199	36,9461	36,8660	36,8773	36,8214	36,7998	36,7997	36,7981
15	54,10	47,28	3900	53,6062	53,8774	53,9513	54,0112	54,0136	54,0522	54,1001	54,1001	54,0981
16	56,60	47,33	4095	56,7532	56,6734	56,6491	56,6405	56,6284	56,6289	56,6001	56,6003	56,5981
RMSE [Mpa]				1,8438	1,1023	0,7393	0,7234	0,7650	0,3245	0,0001	0,0002	0,0023

Tab. 4.13. Summary of $ANN_{(SonReb)}$ data and correlations for on-site concrete strength

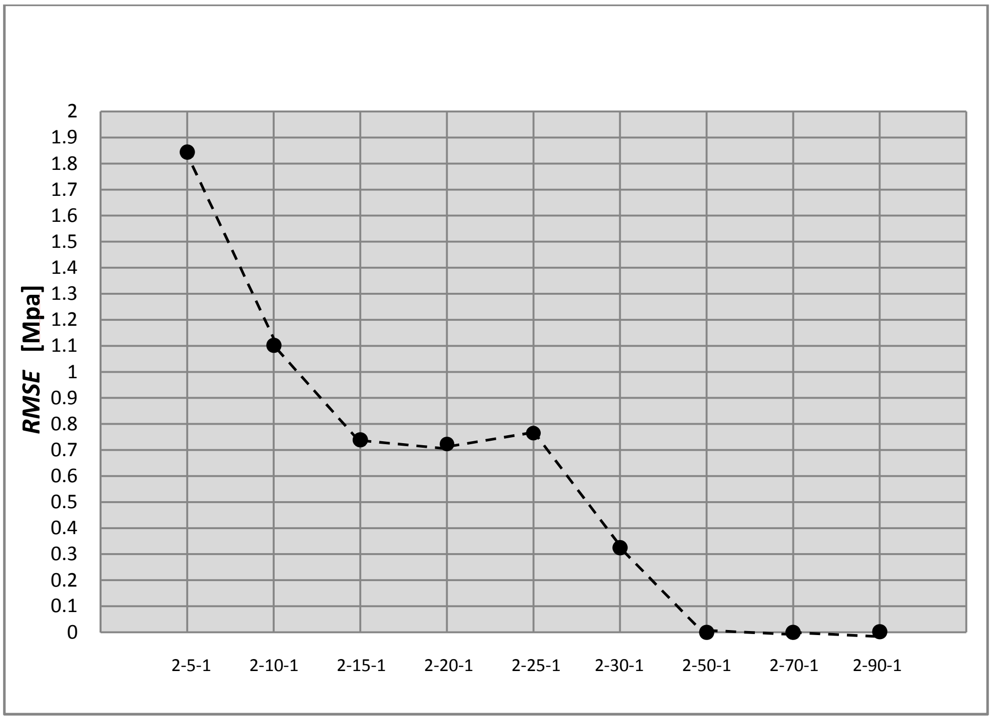


Fig. 4.40. Comparison between the different ANNs performance.

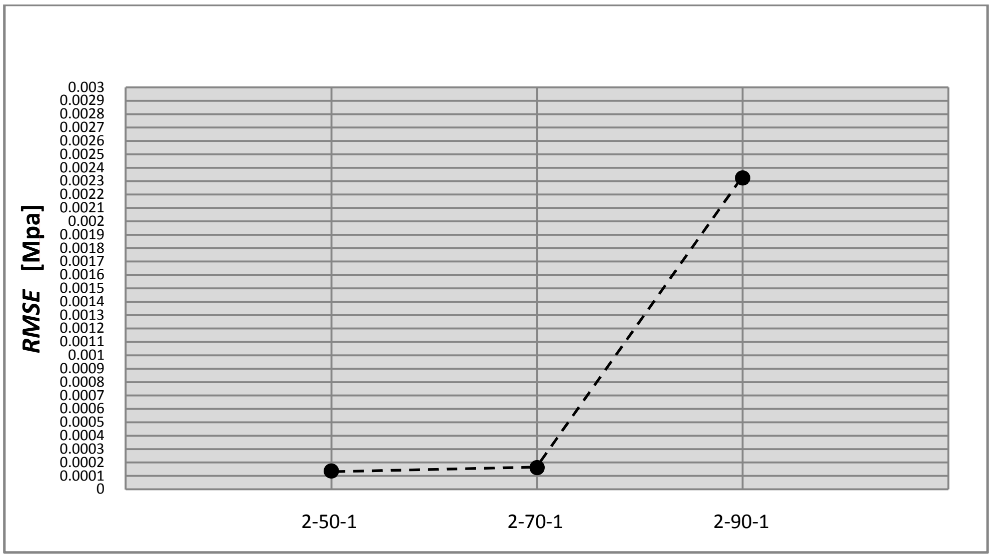


Fig. 4.41. Comparison between the different ANNs performance: particular of the last three ANNs architecture with the most reliability in concrete strength assessment.

ANN$_{(SonReb)}$ 2-5-1

In the following diagrams were represented the training and learning phases of the ANN$_{(SonReb)}$ with 5 neurons in the hidden layer. The principal data of this network is reported in *Tab. 4.14.*

Net Train Param. Epochs	***Best Performance Epochs***	**RMSE [MPa]**
1000	212	1,8438

Tab. 4.14. Summary of 2-5-1 ANN$_{(SonReb)}$ data.

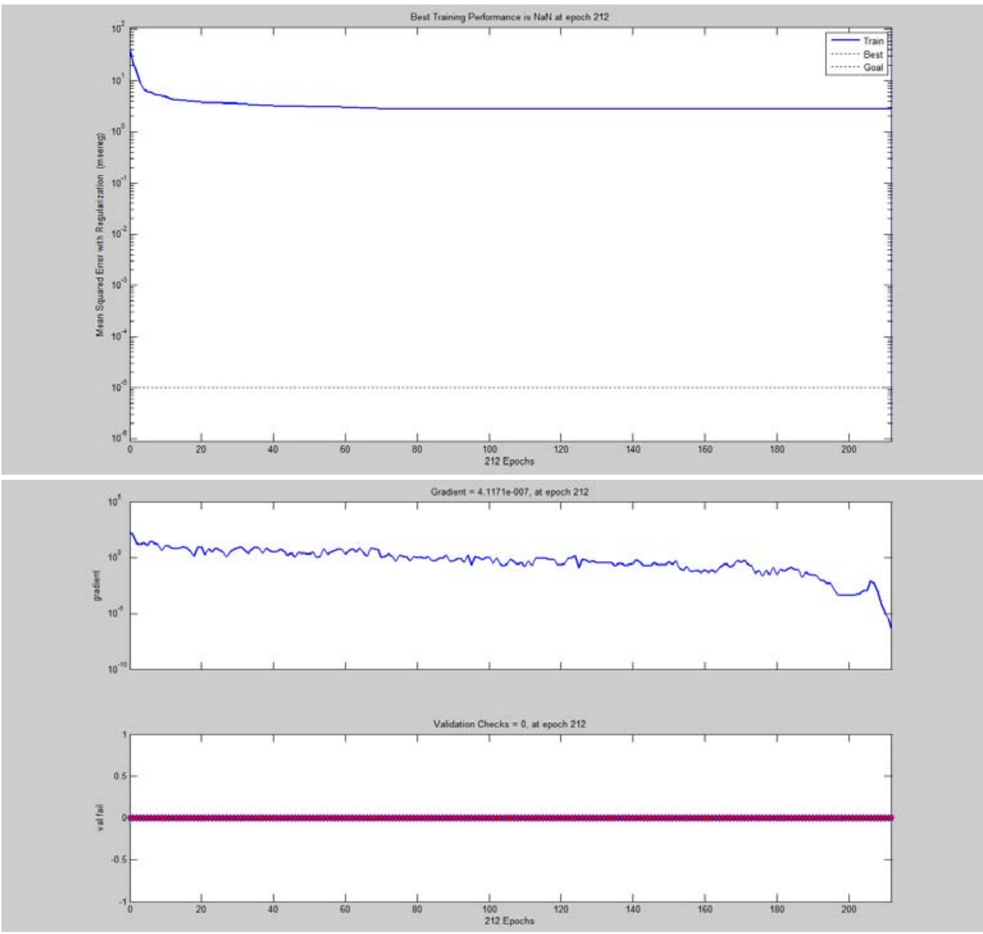


Fig. 4.42. ANN behavior during training performance and gradient descendent.

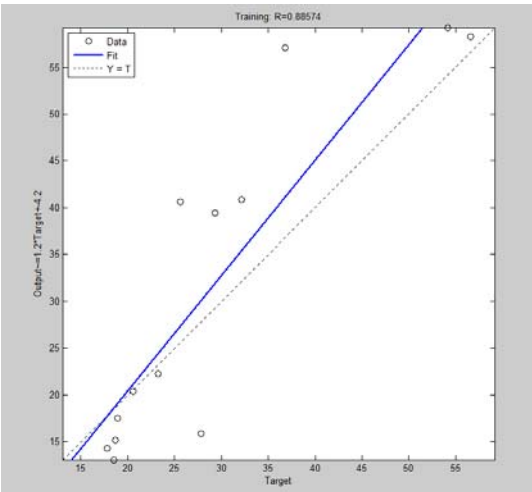


Fig. 4.43. ANN training set.

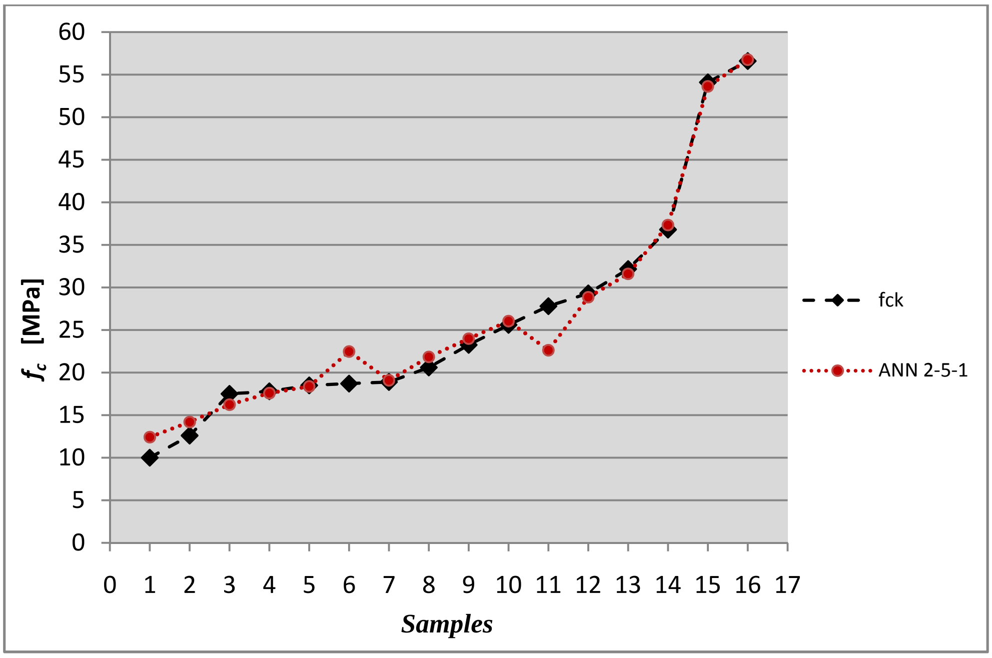


Fig. 4.44. Diagram "f_c – Test number" obtained for ANN$_{(SonReb)}$ 2-5-1

$ANN_{(SonReb)}$ 2-10-1

In the following diagrams were represented the training and learning phases of the $ANN_{(SonReb)}$ with 10 neurons in the hidden layer. The principal data of this network is reported in *Tab. 4.15*.

Net Train Param. Epochs	***Best Performance Epochs***	**RMSE [MPa]**
2000	1063	1,1023

Tab. 4.15. Summary of 2-10-1 $ANN_{(SonReb)}$ data.

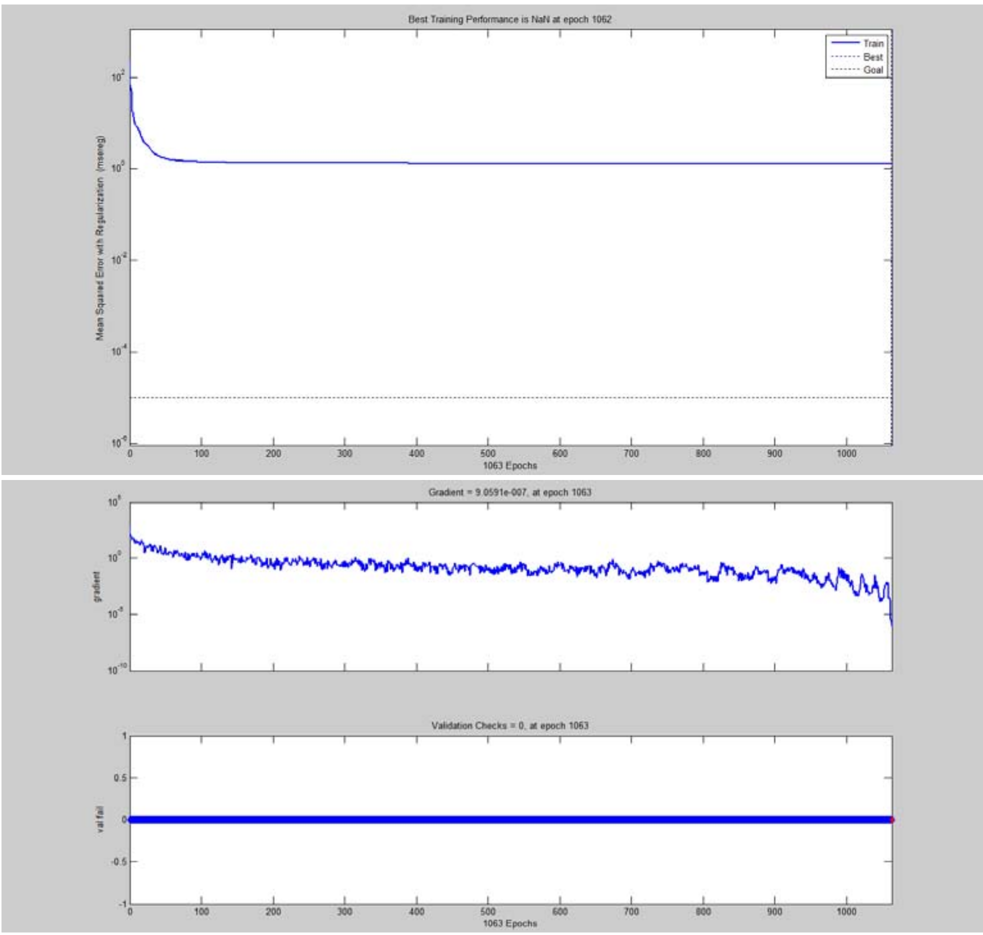


Fig. 4.45. ANN behavior during training performance and gradient descendent.

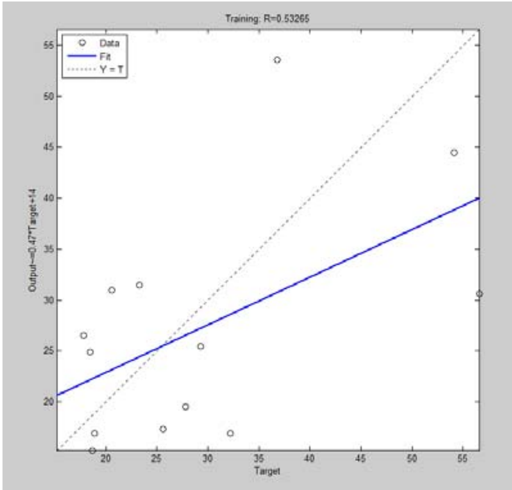


Fig. 4.46. ANN training set.

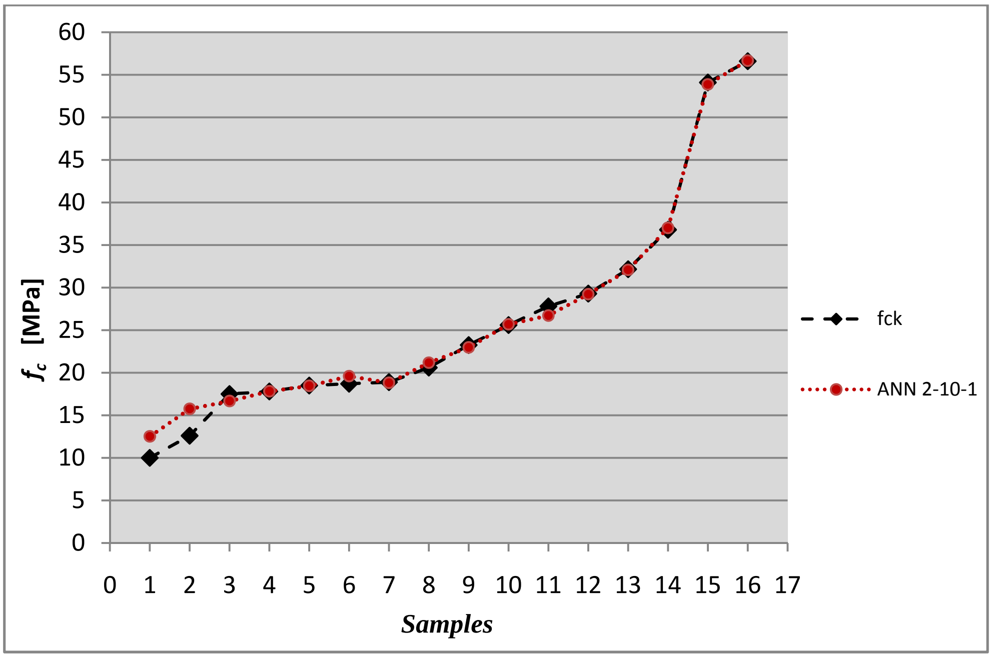


Fig. 4.47. Diagram “f_c – Test number” obtained for ANN$_{(SonReb)}$ 2-10-1

ANN$_{(SonReb)}$ 2-15-1

In the following diagrams were represented the training and learning phases of the ANN$_{(SonReb)}$ with 15 neurons in the hidden layer. The principal data of this network is reported in *Tab. 4.16.*

Net Train Param. *Epochs*	*Best Performance* *Epochs*	**RMSE** **[MPa]**
2000	1935	0,7393

Tab. 4.16. Summary of 2-15-1 ANN$_{(SonReb)}$ data.

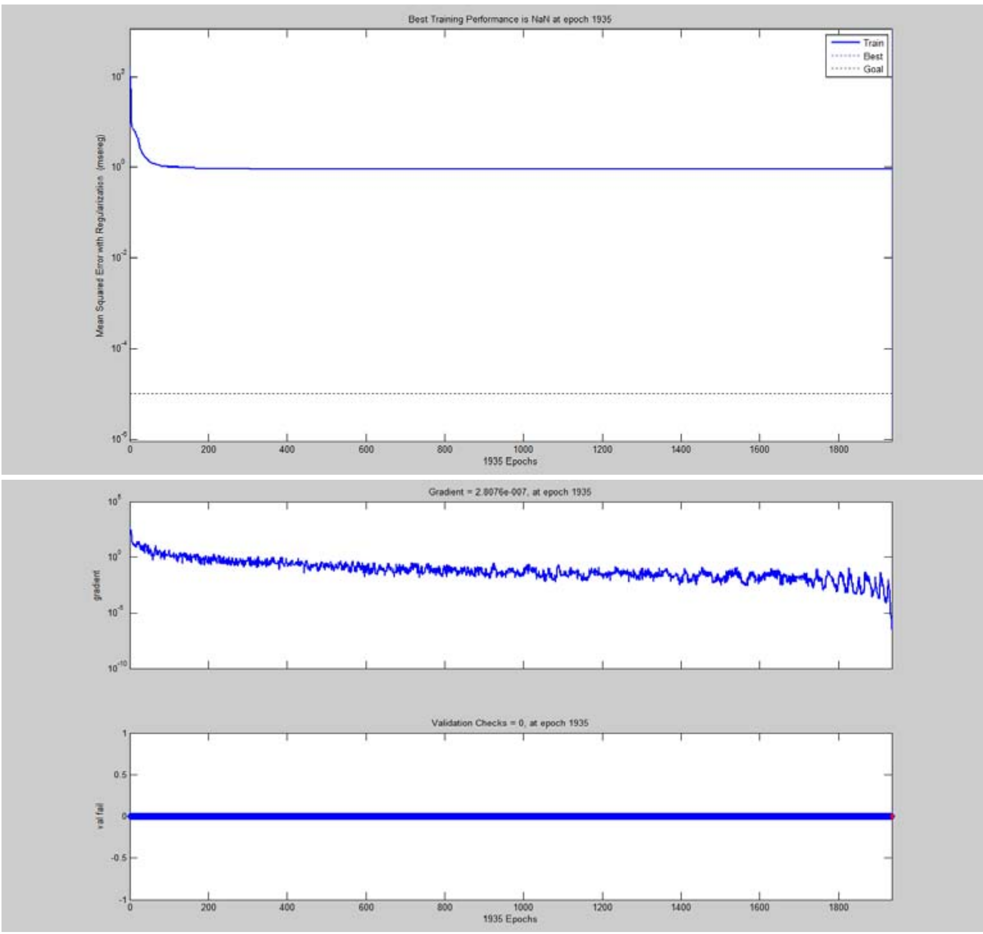


Fig. 4.48. ANN behavior during training performance and gradient descendent.

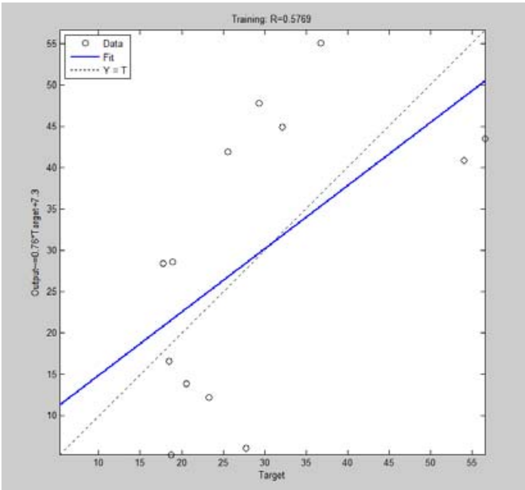


Fig. 4.49. ANN training set.

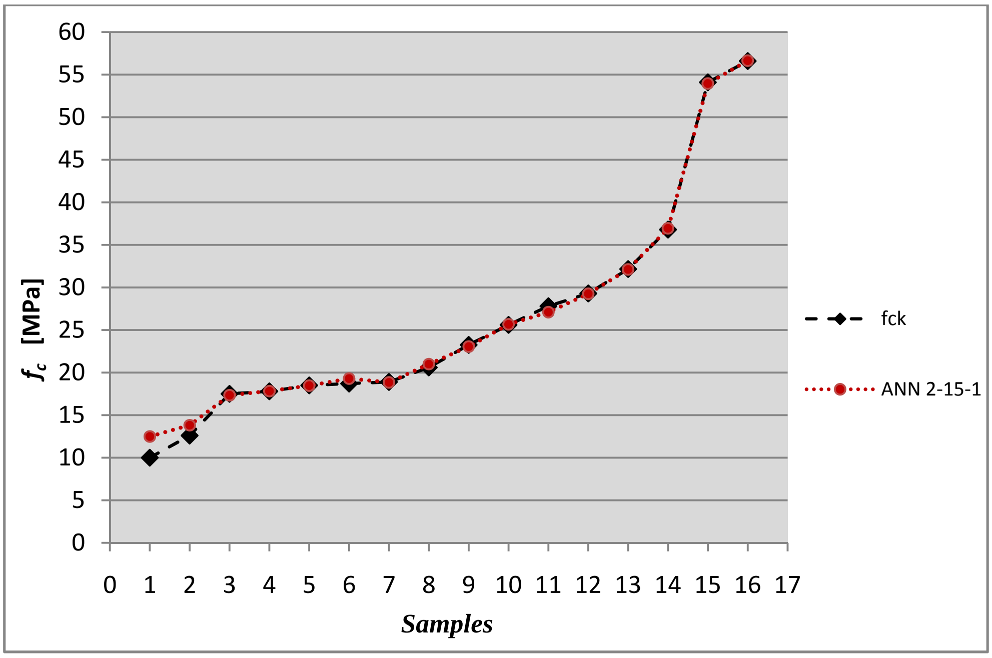


Fig. 4.50. Diagram "f_c – Test number" obtained for $ANN_{(SonReb)}$ 2-15-1

ANN$_{(SonReb)}$ 2-20-1

In the following diagrams were represented the training and learning phases of the ANN$_{(SonReb)}$ with 20 neurons in the hidden layer. The principal data of this network is reported in *Tab. 4.17*.

Net Train Param. Epochs	*Best Performance Epochs*	**RMSE [MPa]**
5000	2109	0,7234

Tab. 4.17. Summary of 2-20-1 ANN$_{(SonReb)}$ data.

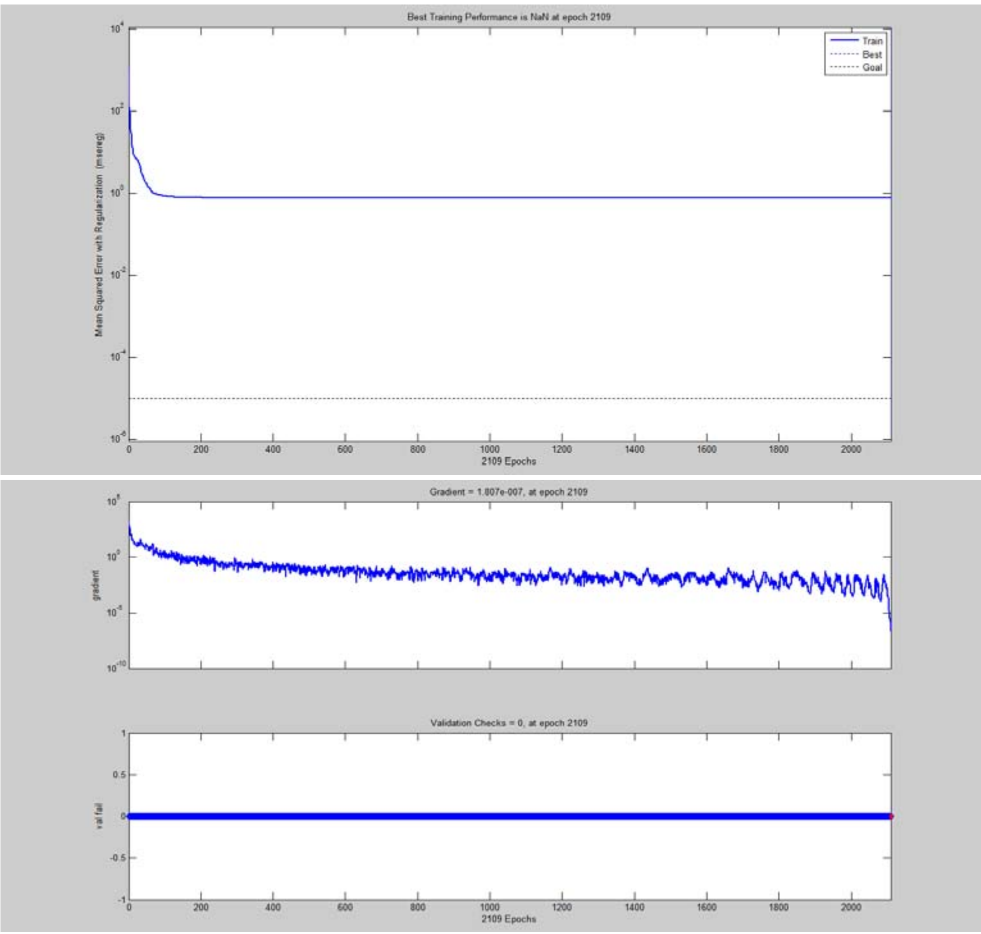


Fig. 4.51. ANN behavior during training performance and gradient descendent.

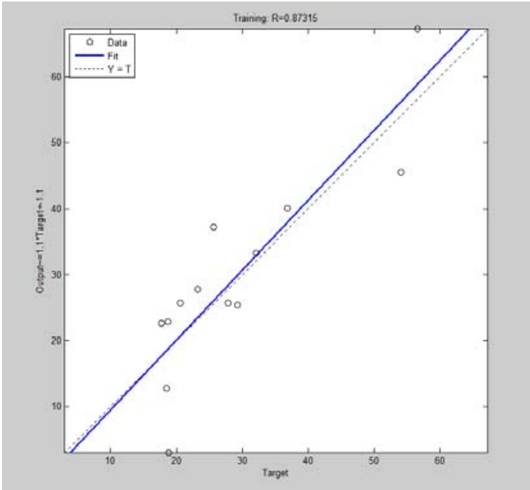


Fig. 4.52. ANN training set.

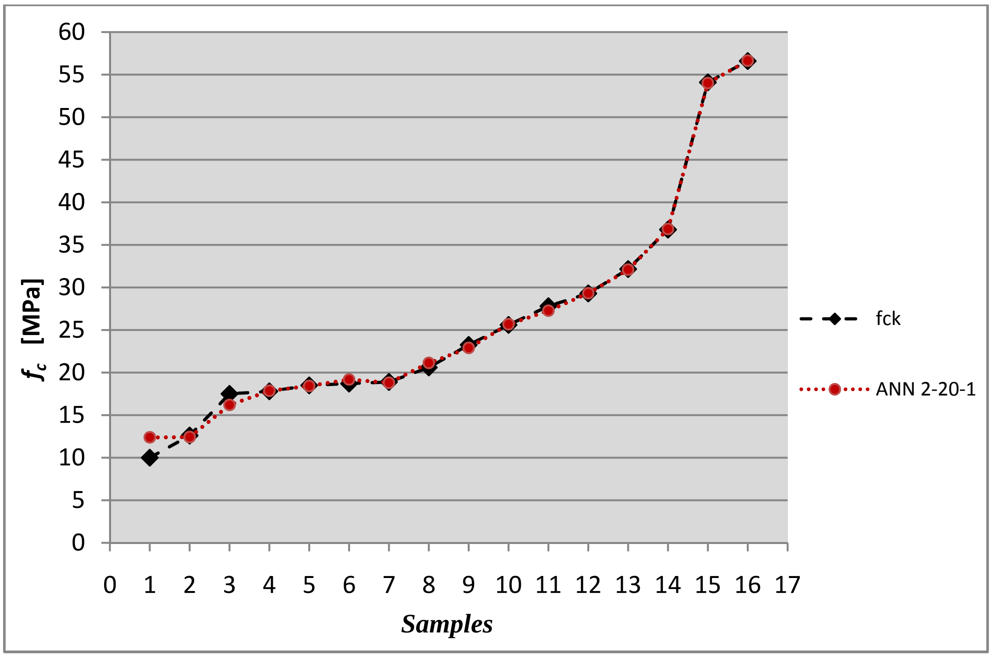


Fig. 4.53. Diagram "f_c – Test number" obtained for ANN$_{(SonReb)}$ 2-20-1

ANN$_{(SonReb)}$ 2-25-1

In the following diagrams were represented the training and learning phases of the ANN$_{(SonReb)}$ with 25 neurons in the hidden layer. The principal data of this network is reported in *Tab. 4.18.*

Net Train Param. Epochs	***Best Performance Epochs***	**RMSE [MPa]**
8000	3842	0,7650

Tab. 4.18. Summary of 2-25-1 ANN$_{(SonReb)}$ data.

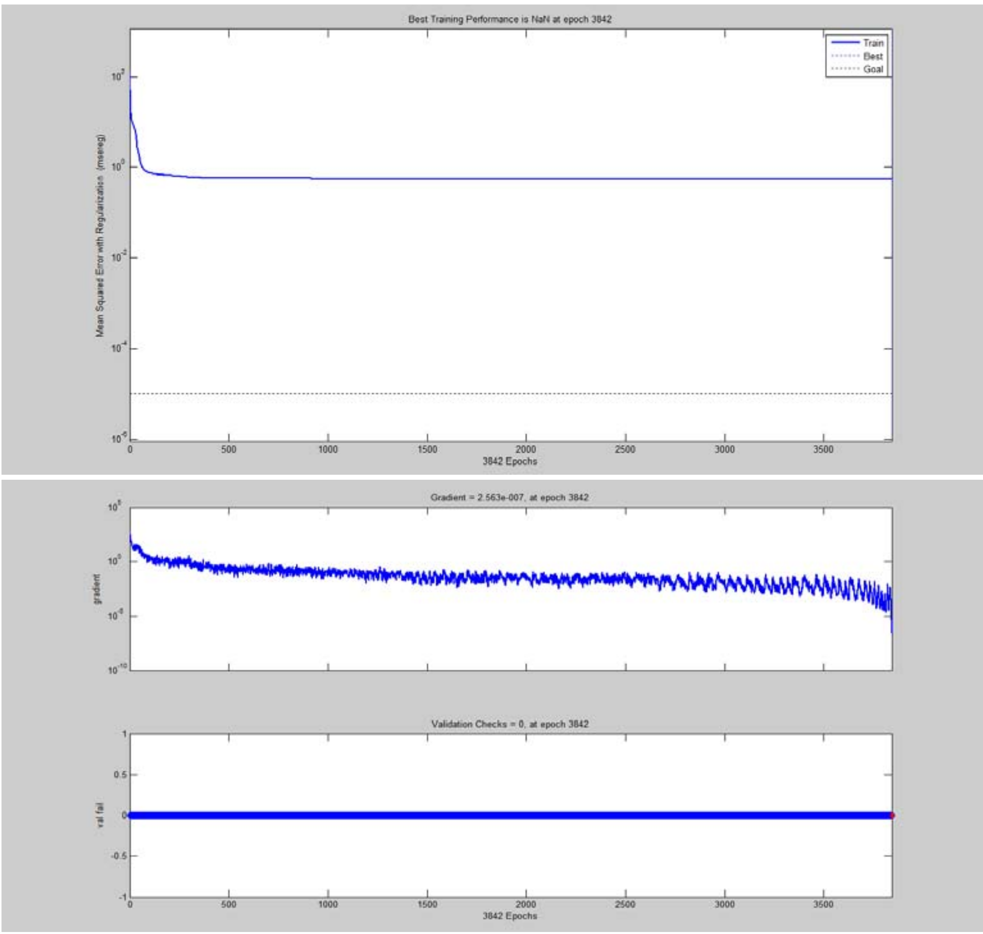


Fig. 4.54. ANN behavior during training performance and gradient descendent.

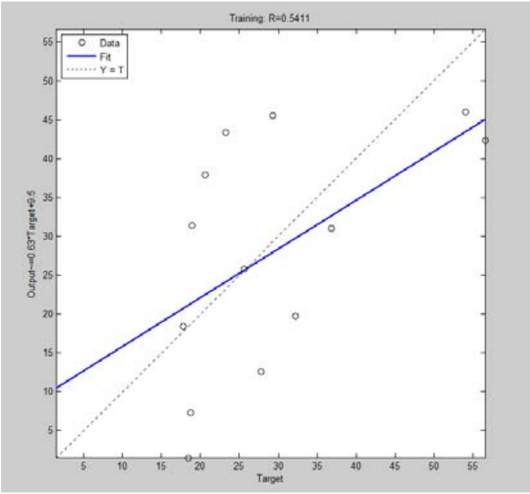


Fig. 4.55. ANN training set.

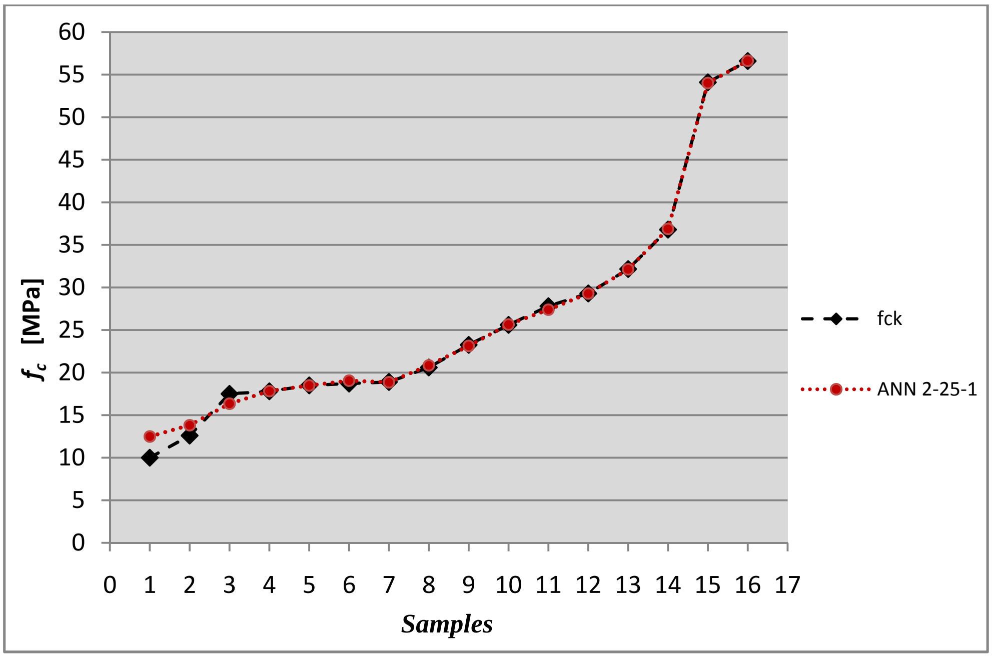


Fig. 4.56. Diagram "f_c – Test number" obtained for $ANN_{(SonReb)}$ 2-25-1

ANN(SonReb) 2-30-1

In the following diagrams were represented the training and learning phases of the ANN(SonReb) with 30 neurons in the hidden layer. The principal data of this network is reported in *Tab. 4.19.*

Net Train Param. Epochs	***Best Performance Epochs***	**RMSE [MPa]**
8000	3953	0,3245

Tab. 4.19. Summary of 2-30-1 ANN(SonReb) data.

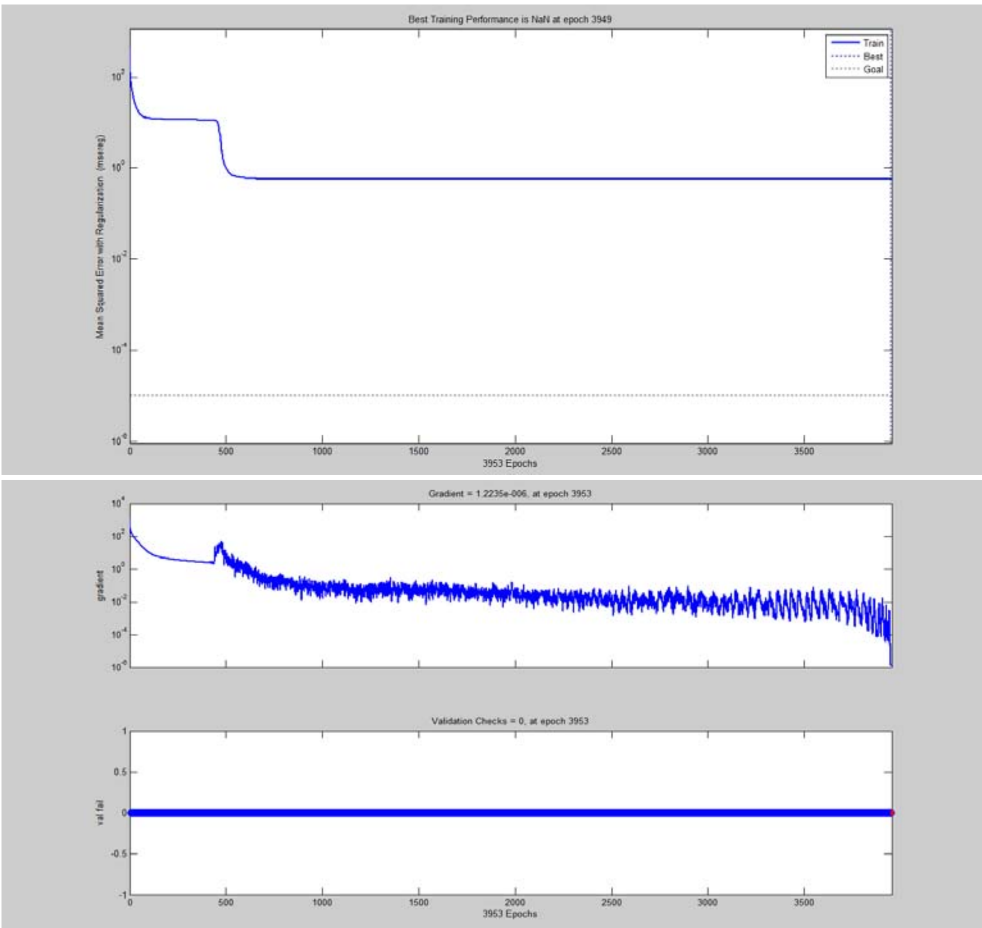


Fig. 4.57. ANN behavior during training performance and gradient descendent.

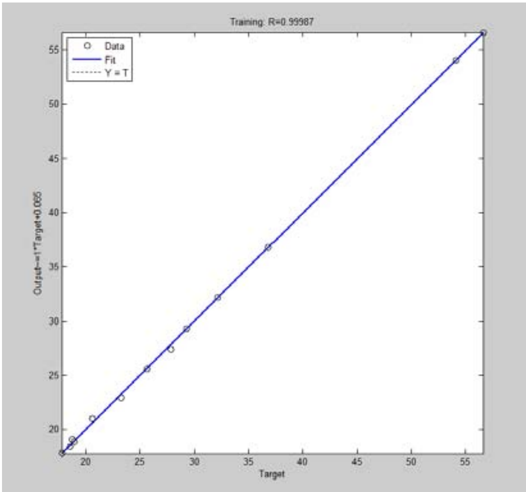


Fig. 4.58. ANN training set.

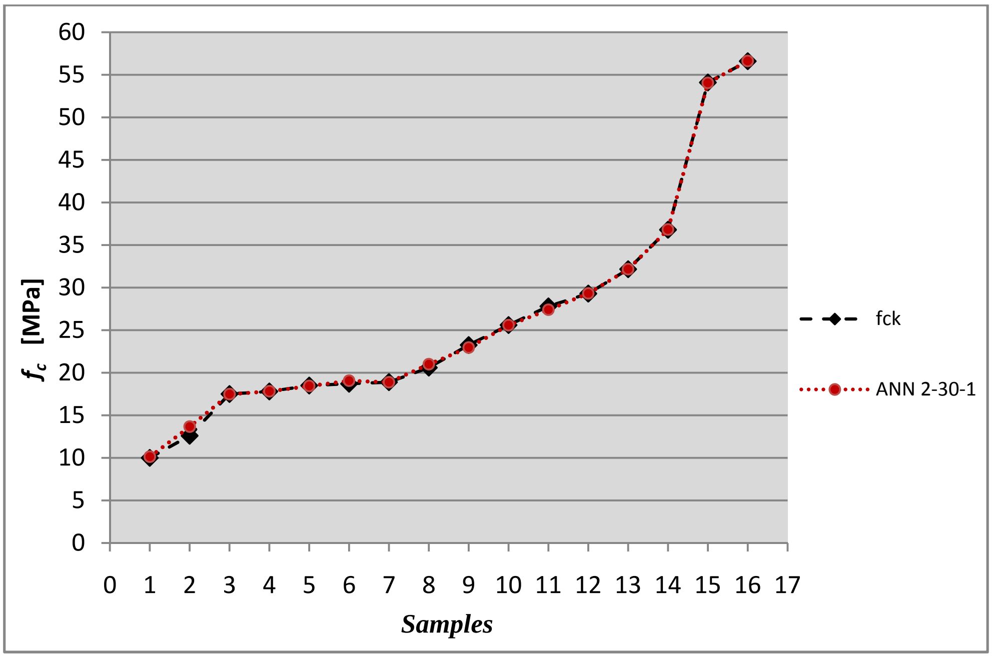


Fig. 4.59. Diagram "f_c – Test number" obtained for $ANN_{(SonReb)}$ 2-30-1

ANN$_{(SonReb)}$ 2-50-1

In the following diagrams were represented the training and learning phases of the ANN$_{(SonReb)}$ with 50 neurons in the hidden layer. The principal data of this network is reported in *Tab. 4.20.*

Net Train Param. Epochs	***Best Performance Epochs***	**RMSE [MPa]**
10000	6074	0,001

Tab. 4.20. Summary of 2-50-1 ANN$_{(SonReb)}$ data.

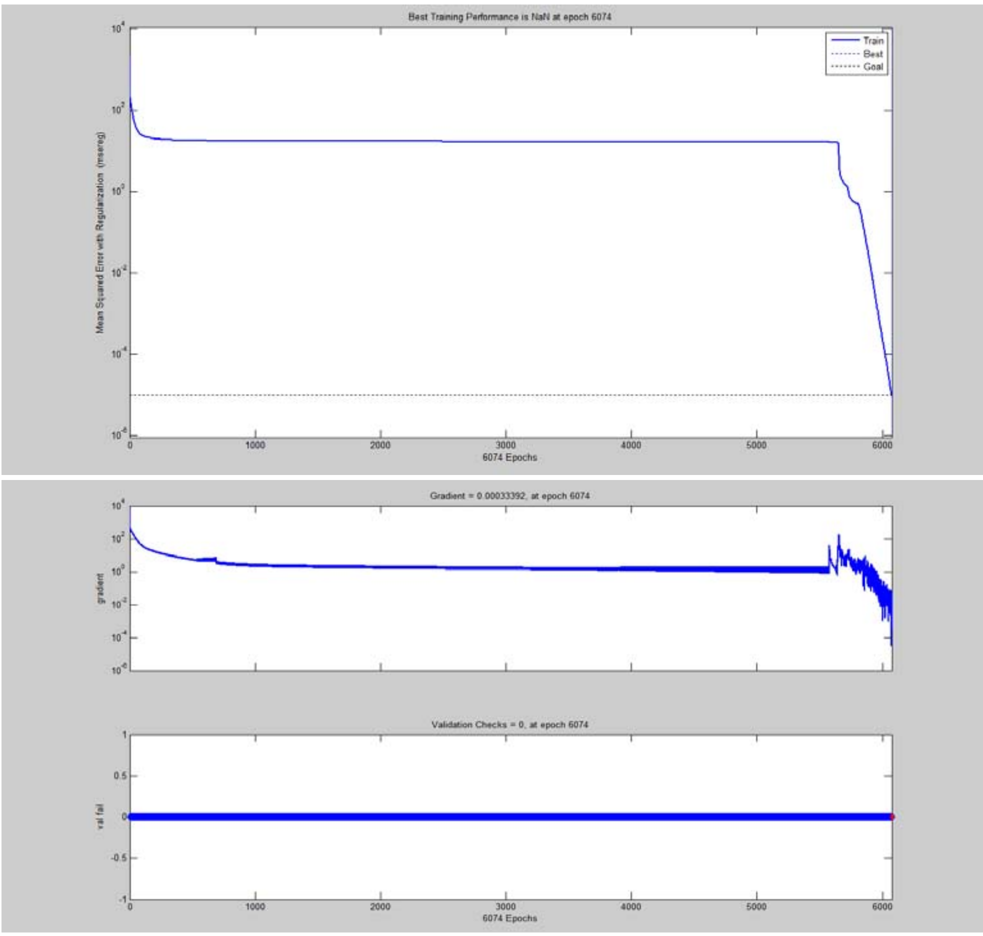


Fig. 4.60. ANN behavior during training performance and gradient descendent.

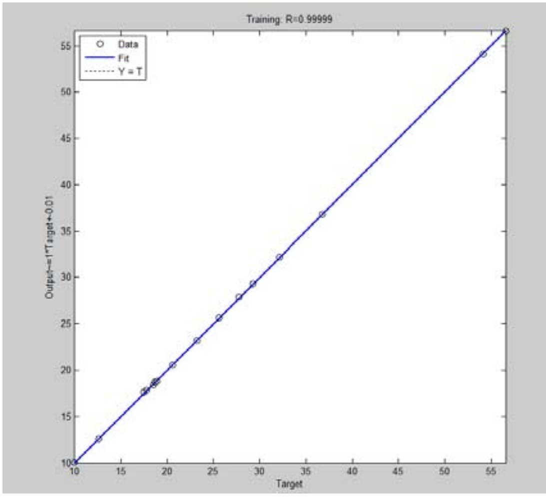


Fig. 4.61. ANN training set.

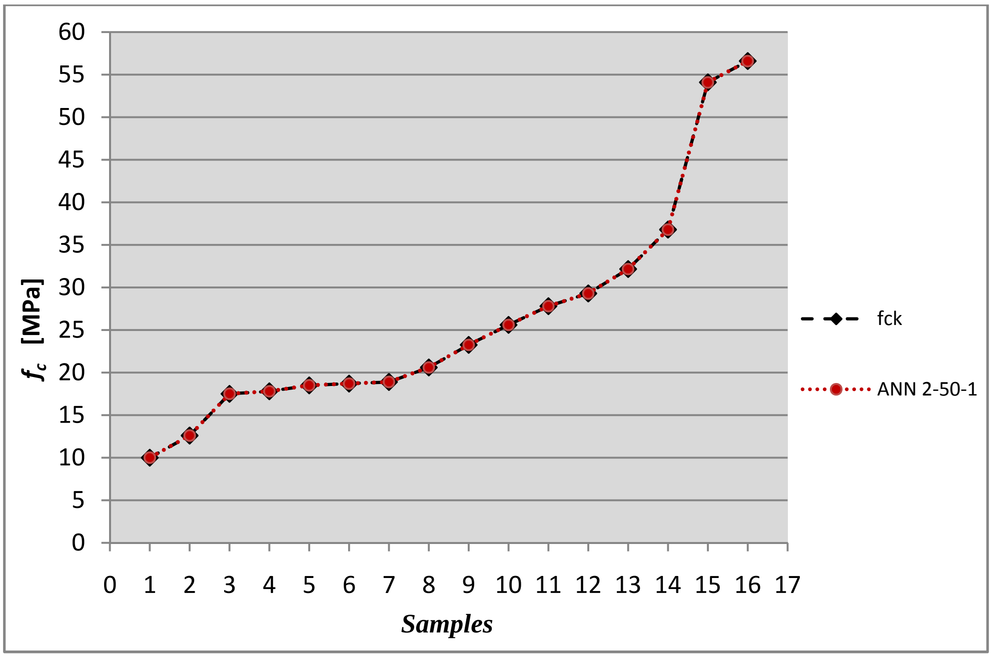


Fig. 4.62. Diagram “f_c – Test number” obtained for ANN$_{(SonReb)}$ 2-50-1

***ANN$_{(SonReb)}$* 2-70-1**

In the following diagrams were represented the training and learning phases of the ANN$_{(SonReb)}$ with 70 neurons in the hidden layer. The principal data of this network is reported in *Tab. 4.21*.

Net Train Param. Epochs	***Best Performance Epochs***	**RMSE [MPa]**
200000	101626	0,002

Tab. 4.21. Summary of 2-70-1 ANN$_{(SonReb)}$ data.

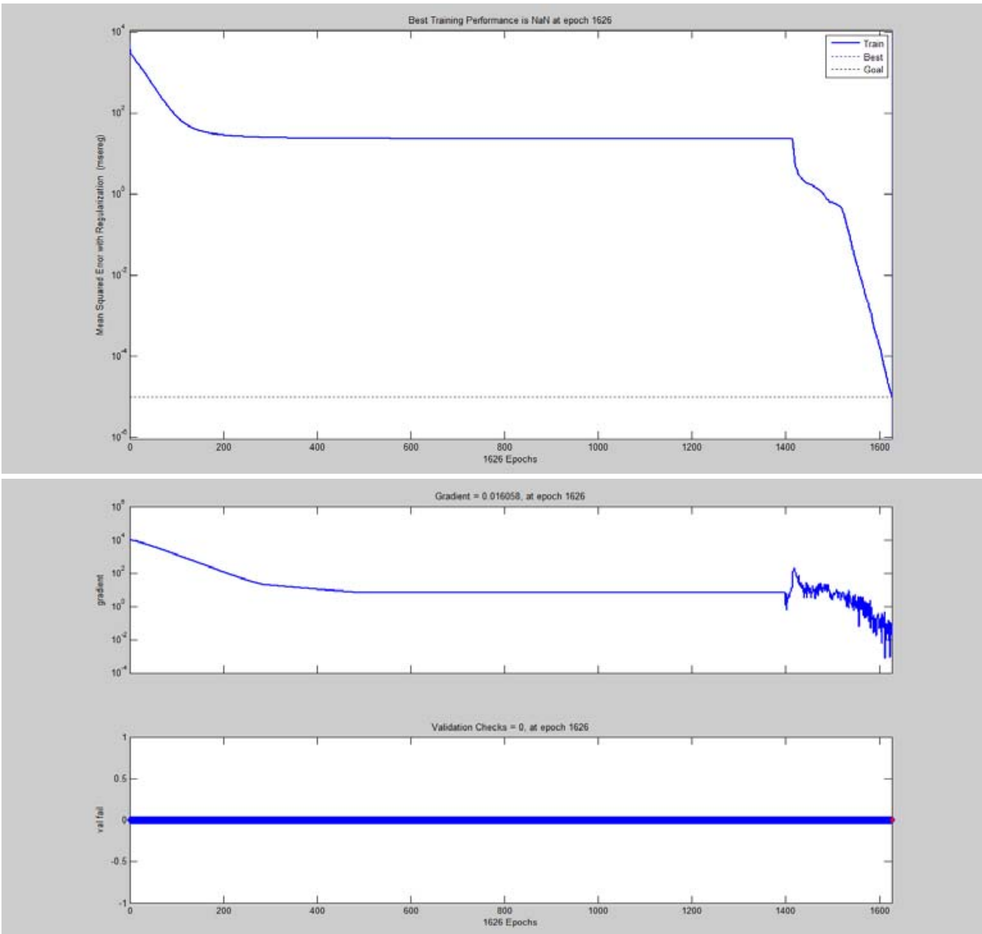


Fig. 4.63. ANN behavior during training performance and gradient descendent.

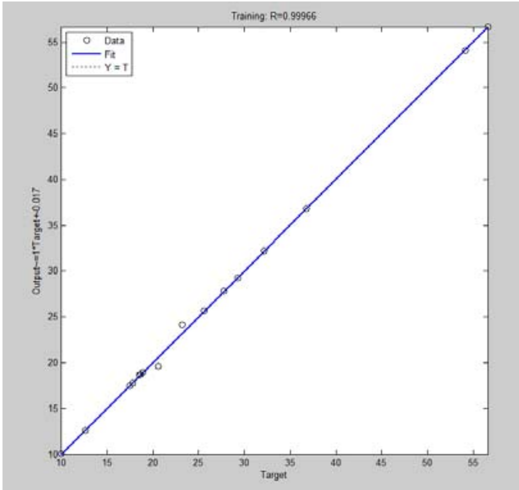


Fig. 4.64. ANN training set.

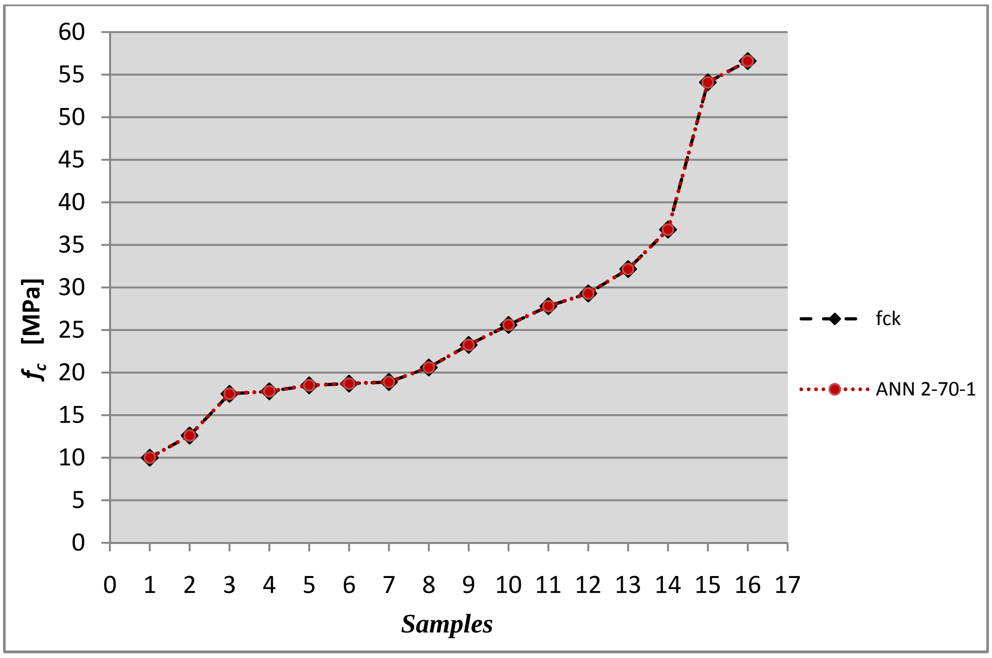


Fig. 4.65. Diagram "f_c – Test number" obtained for ANN$_{(SonReb)}$ 2-70-1

ANN$_{(SonReb)}$ 2-90-1

In the following diagrams were represented the training and learning phases of the ANN$_{(SonReb)}$ with 90 neurons in the hidden layer. The principal data of this network is reported in *Tab. 4.22.*

Net Train Param. Epochs	***Best Performance Epochs***	**RMSE [MPa]**
1000000	244198	0,003

Tab. 4.22. Summary of 2-90-1 ANN$_{(SonReb)}$ data.

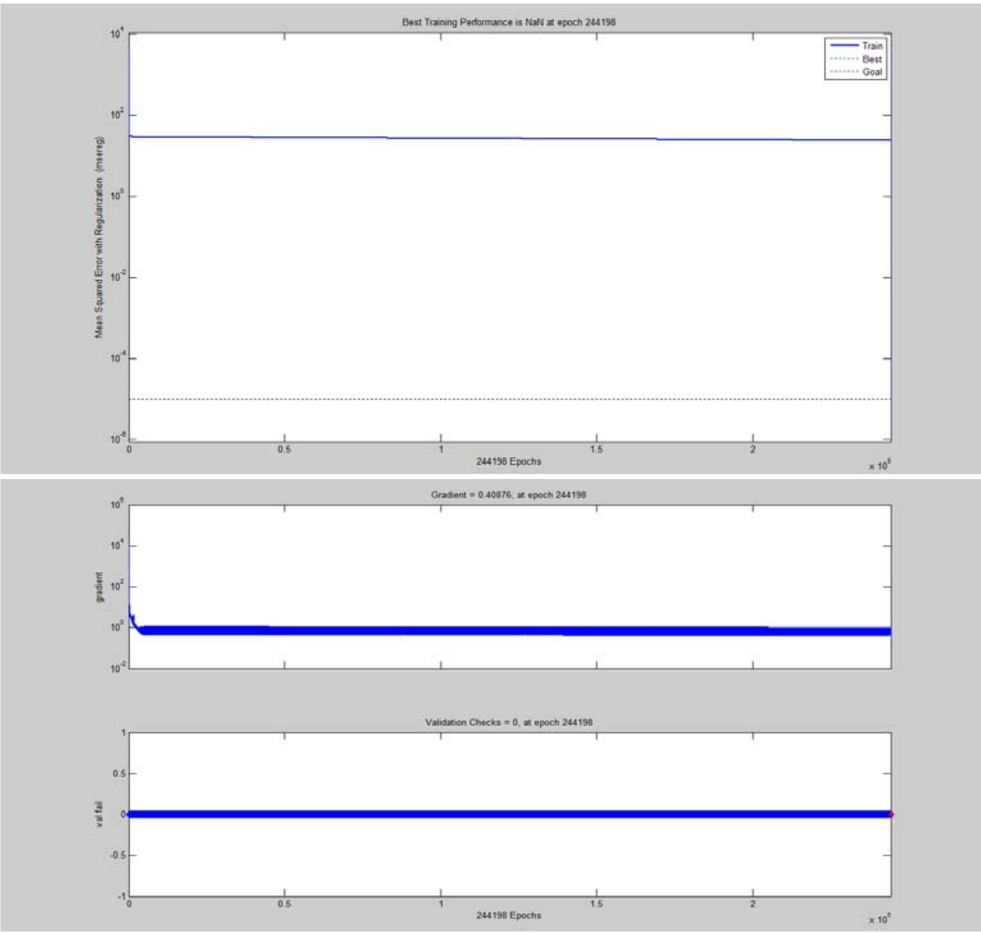


Fig. 4.66. ANN behavior during training performance and gradient descendent.

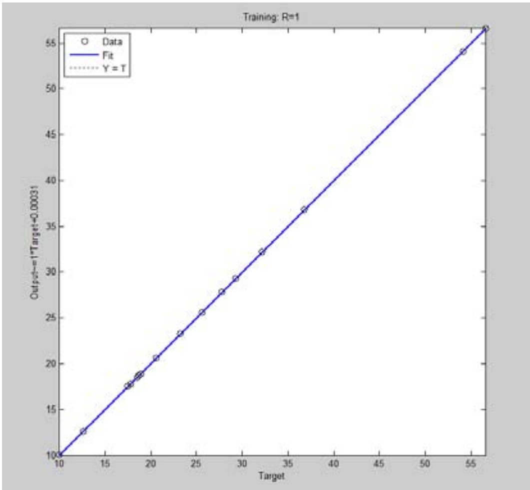


Fig. 4.67. ANN training set.

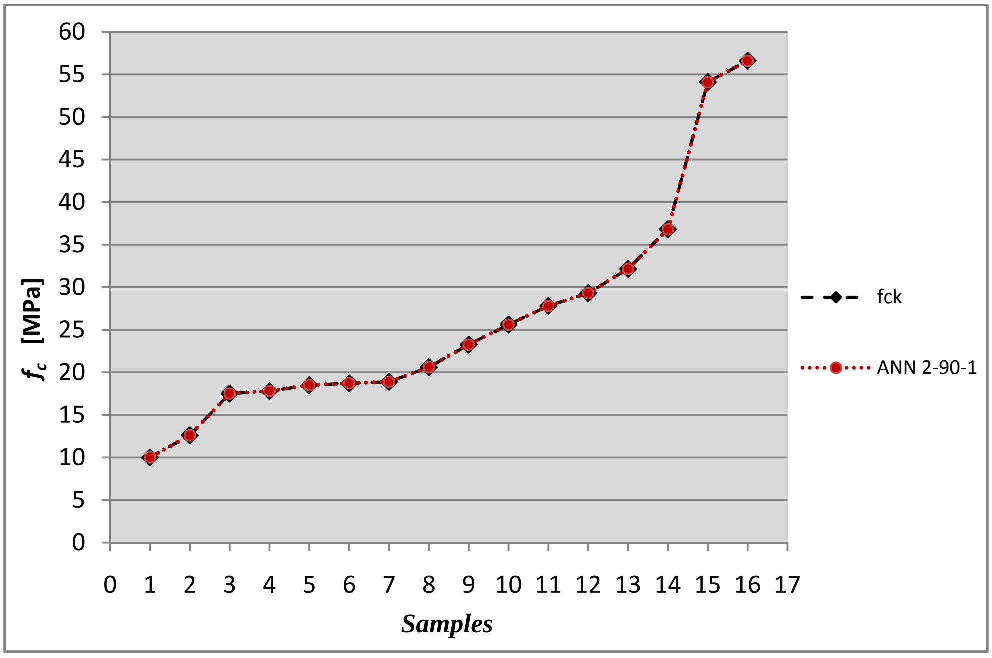


Fig. 4.68. Diagram "f_c – Test number" obtained for ANN$_{(SonReb)}$ 2-90-1

Conclusions

In general, the results obtained from the study indicate the excellent estimation potential of a multilayer feed-forward neural network trained with *backpropagation error algorithm* in the evaluation of concrete compressive strength, starting only from the non-destructive parameters.

In fact, the root mean squared error (*RMSE*) values obtained for both cases are to be considered reasonably low in relation to those obtained with the formulas in the literature, thus indicating the accuracy of the estimations. The determination of the *RMSE* has also indicated for the two types of neural networks for the best solution in terms of number of neurons of the hidden layer depending on the result to be achieved and the computational time to reach it.

With regard to the artificial neural networks characterized by a single input parameter (*UPV*), a considerable improvement (*RMSE* decreases from 9,672MPa to 1,9786MPa) in the estimation of the concrete compressive strength has reached for networks that employ a large number of neurons in the hidden layer. This behavior depends on the fact that the processes of the network is derived from a single input parameter and for this reason a greater computational work in data processing is necessitated by the system. Thus, in order to ensure the workload required a greater amount of neurons in the hidden layer are needed.

Analyzing the results obtained for the neural networks characterized by two input parameters (i.e. *RI* and *UPV*) it can be noted a significant improvement in the estimation of concrete compressive strength since for networks with 50 neurons into the hidden layer (*RMSE* = 0,00013MPa, about 50000 times lower than that obtained with the regression formulas found in the literature). Therefore in the first type of ANNs the relationship between input and output parameters is unable to overcome the limitations of the single non-destructive analysis (ultrasonic pulse velocity test). This phenomenon is only partially overcome by growing the number of neurons, which creates a consequent increase in the number of connections, and then the processing of the system. The almost total overcome of this limitation occurs when the neural networks have two or more independent input parameters. In this case, in fact, similarly to as occurs in the combined method SonReb, the limitations of the individual input parameters in relation to the mechanical characteristic to be determined are attenuated to each other.

We can therefore conclude that the analyzes carried out on artificial neural networks have shown their full potential by demonstrating the mismatch of the report: "*greater number of neurons in the hidden layer implies better computational results*" if the input parameters of the network are varied and well assorted from the point of view of the physical characteristics analyzed.

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