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EXPERIMENTAL INVESTIGATION ON THE SEISMIC PERFORMANCE OF A MULTI-COMPONENT SYSTEM FOR MAJOR-HAZARD INDUSTRIAL FACILITIES

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ABSTRACT

Past earthquakes demonstrated the high vulnerability of industrial facilities equipped with complex process technologies leading to serious damage of the process equipment and multiple and simultaneous release of hazardous substances in industrial facilities. Nevertheless, the design of industrial plants is inadequately described in recent codes and guidelines, as they do not consider the dynamic interaction between the structure and the installations and thus the effect of seismic response of the installations on the response of the structure and vice versa. The current code-based approach for the seismic design of industrial facilities is considered not enough for ensure proper safety conditions against exceptional event entailing loss of content and related consequences. Accordingly, SPIF project (Seismic Performance of Multi-Component Systems in Special Risk Industrial Facilities) was proposed within the framework of the European H2020 - SERA funding scheme (Seismology and Earthquake Engineering Research Infrastructure Alliance for Europe). The objective of the SPIF project is the investigation of the seismic behaviour of a representative industrial structure equipped with complex process technology by means of shaking table tests. The test structure is a three-story moment resisting steel frame with vertical and horizontal vessels and cabinets, arranged on the three levels and connected by pipes. The

dynamic behaviour of the test structure and of its relative several installations is investigated. Furthermore, both process components and primary structure interactions are considered and analyzed. Several PGA-scaled artificial ground motions are applied to study the seismic response at different levels. After each test, dynamic identification measurements are carried out to characterize the system condition. The contribution presents the experimental setup of the investigated structure and installations, selected measurement data and describes the obtained damage. Furthermore, important findings for the definition of performance limits, the effectiveness of floor response spectra in industrial facilities will be presented and discussed.

Keywords: industrial facilities; piping; installations; seismic loading; earthquakes; shaking-table

NOMENCLATURE

BFJ	Bolt Flanged Joint
FRF	Frequency Response Function
FRS	Floor Response Spectra
FEM	Finite Element Method
LoC	Loss of Containment
MRF	Moment Resisting Frame

NC	Near Collapse limit state
OBE	Operating Basis Earthquake Condition Limit State
OP	Operational limit state
PBEE	Performance-Based Earthquake Engineering
PSD	Power Spectral Density
SSE	Safe Shutdown Earthquake

1. INTRODUCTION

The impact of a natural disaster on facility storing or processing dangerous substances can result in the release of those hazardous substances with possible severe off-site consequences through toxic-release, fire or explosion scenarios. Accidents triggered by a natural hazard, involving release of dangerous substances are commonly referred to as NaTech events, [1]. These include releases from fixed chemical installations and spills from oil and gas pipelines. One of the main issues of NaTech accidents is the simultaneous occurrence of a natural disaster and a technological accident; both require simultaneous efforts in dealing with a situation in which lifelines designed for a disaster mitigation are likely unavailable, as they may have been damaged by the natural disaster. NaTech events derives from the interaction between industrial and natural hazards. In particular, earthquakes have been recognized as one of the most disruptive natural hazard, as demonstrated by many past earthquake [2], which raised public concern because of the general unpreparedness of the countries in predicting effects and consequences in the aftermath of a disaster [3].

The objective of this work is the simulated investigation of the seismic behaviour of industrial plants equipped with complex process technology by means of shaking table tests [4]. The necessity exists, since past earthquakes have shown that critical interactions can occur in these plants between the supporting structure and plant components, but also between the individual plant components. The structural or process-related interactions can lead to serious secondary damages which, in addition to loss of production, also pose a danger to humans and the environment if hazardous substances are released due to leakages. Along this main vein and in a performance-based design perspective, the issues investigated in depth are the interaction between a primary moment resisting frame (MRF) steel structure and secondary process components that influence the performance of the whole system; and a proper check of floor spectra predicted by standards and specialised literature. Experimental data clearly show a favourable performance of the MRF structure, some weaknesses of local details due to the interaction between floor crossbeams and process components and, finally, the overconservatism of current design standards w.r.t. floor spectra predictions [5]-[6]-[7]. Thus, the present paper, gathers the main characteristics of an archetype industrial facility, the test programme, and main results.

2. DESCRIPTION OF THE MOCK-UP

The full scale mock-up consists of a primary steel structure supporting horizontal and vertical tanks, piping installations and a cabinet. Furthermore, a conveyor was placed on the reinforced concrete slab without any connection to the main structure. It was conceived as a three-storey steel frame with flexible diaphragms made of crossbeams hinged to the frame beams.

Figure 1 shows the test structure on the shaking table whilst Figure 2 depicts the three dimensional CAD model. The ground-plan dimensions are 3.7 m x 3.7 m and the storey height is 3.1 m, which leads to a total height of 9.3 m. The columns were fully clamped to a reinforced concrete base plate. This latter has dimensions of 4.8 m x 4.8 m and a thickness of 0.4 m and was constructed with a concrete of class C30/37. The horizontal load bearing system of the steel structure consisted of two moment resisting frames (MRFs) in the direction of the seismic excitation and two braced frames in the transverse direction equipped with tension/compression bracings with circular cross section. These latter were used in order to limit lateral movements and torsional effects. The two MRFs were connected through crossbeams, which are fastened to the frame beams by simple bolted connections with web stiffeners. These crossbeams served in turn as bearing supports for bearing platforms which were installed with bolted connections to place the secondary elements.

In total four tanks were installed, two vertical tanks on the first level and two horizontal tanks on the second level. Furthermore, one electrical cabinet was placed on the first level. The nominal pipe diameters for the pipes connecting pipes of the tanks are DN 100. The suspended pipes on the third story will be installed with DN 80. Moreover, bolted flange joints BFJs were necessary in order to connect the several parts of pipe. More precisely, the loose flanges adjacent to the tanks were conceived as moving flanges, while the others were simple flanges welded to pipes.

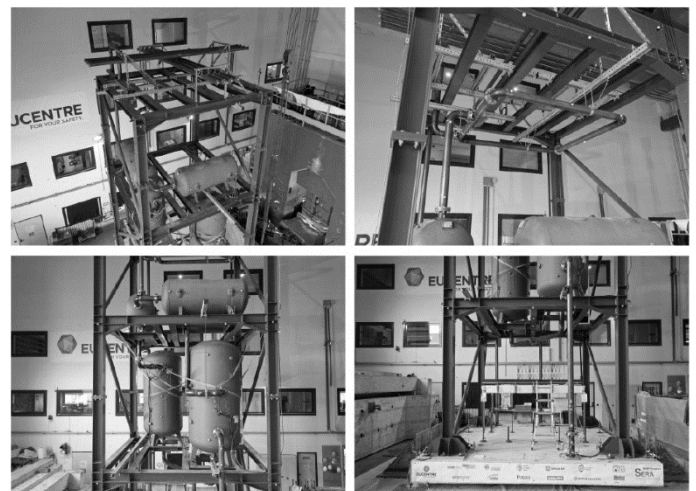


FIGURE 1: MOCK-UP PHOTOGRAPHS AT THE EUCENTRE LABORATORY

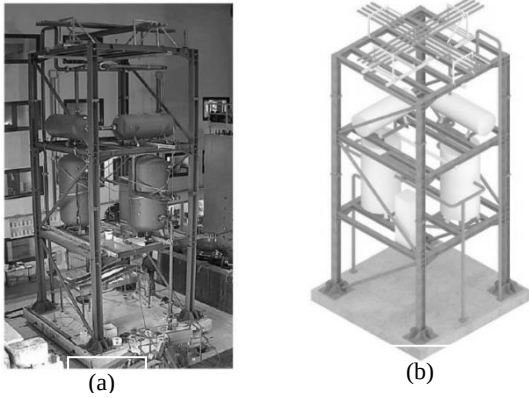


FIGURE 2: SPIF MOCK-UP: (a) SITE CONSTRUCTION AND (b) 3D VIEW OF THE NUMERICAL SPIF MODEL.

2.1 Monitoring System and Layout

A series of sensors was positioned on both the primary structure and secondary units. The instrumentation consists of accelerometers, displacement transducers (LVDTs) and strain gauges (SGs). The distribution of the sensors is summarized in Table 1 and represented at each floor in Figures 3-4.

TABLE 1: SENSORS TYPE AND NUMBERS IN THE MOCK-UP

Level	Accelerometer	LVDT	Strain gauges
Floor 0	6	15	12
Floor 1	16	-	-
Floor 2	11	-	-
Floor 3	7	-	-
Roof top	14	4	-
Pipes (Pos. 1, 4, 5, 6)	-	12	12

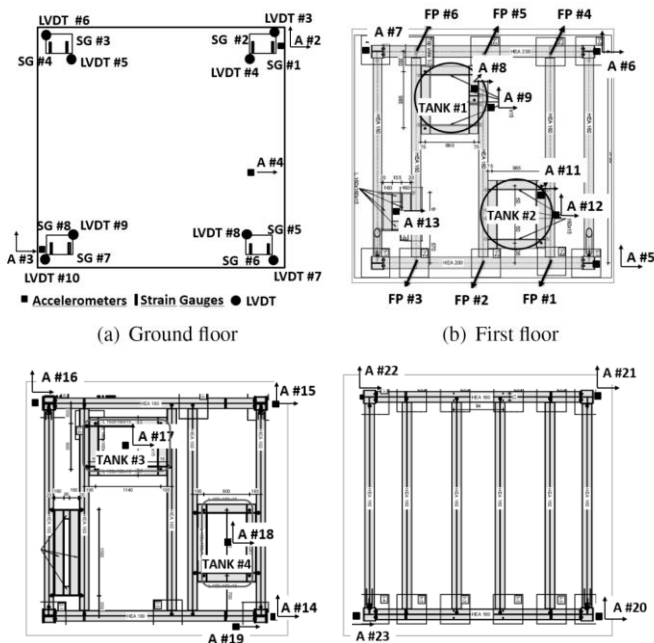
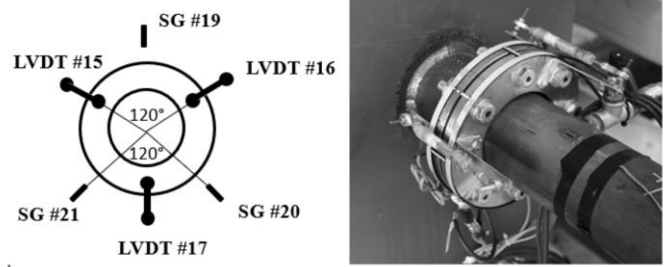


FIGURE 3: SENSORS LOCATION AT EACH FLOOR.

In particular, in SPIF structure have been used 54 uniaxial



(a) BFJ Pos #4 - Top and side view

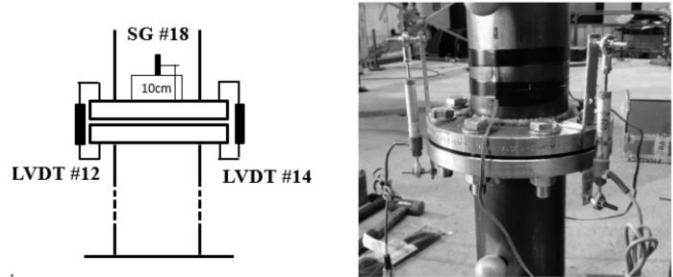


FIGURE 4: PIPING MONITORING: SENSORS LOCATION ON THE BFJS.

accelerometers placed in both X and Y plane direction and on each level in order to better describe response in frequency of the prototype industrial plant.

Besides, 16 LVDTs and 8 strain gauges have been applied for standard checks on the response of the steel frame structure. Moreover, a system of FBG fiber optic sensors have been applied to critical flanges in the SPIF structure in order to detect the possibility of leakage and loss of containment.

A displacement sensor setup was also introduced with an optical measurement system, made of 78 markers applied to different points of both the primary structure and secondary elements, as indicated in Fig. 5: a tracking system of 16 cameras equipped with an infrared sensor for movement recording has therefore been installed on the mock-up.

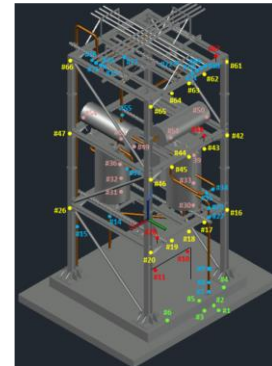


FIGURE 5: MARKERS LAYOUT.

3. DESIGN OF THE EXPERIMENTAL TEST

Sequence and intensity levels of shaking table tests are performed so as to reach more severe damages or even collapses in components by passing from fully operational (OP) to near collapse (NC) limit states, or, as in case of major industrial plants, from operating condition or design basis earthquake condition (OBE) to safe shutdown earthquake condition (SSE).

In particular, the primary structure has been designed in order to remain in linear-elastic domain behaviour, while the secondary structural and non-structural elements are designed according to the state of the arts and standard codes.

3.1 Input selection

The applied seismic action was based on a linear elastic response spectrum Type 1, with soil condition C, a PGA=0.6 g and a damping ratio of 5 % in agreement with Eurocode 8-Part 1 [8]. This spectrum, depicted in Figure 6a, exhibits an initial spectral acceleration of 0.63 g and a plateau range between the periods 0.2 s and 0.6 s with a spectral acceleration of 1.725 g.

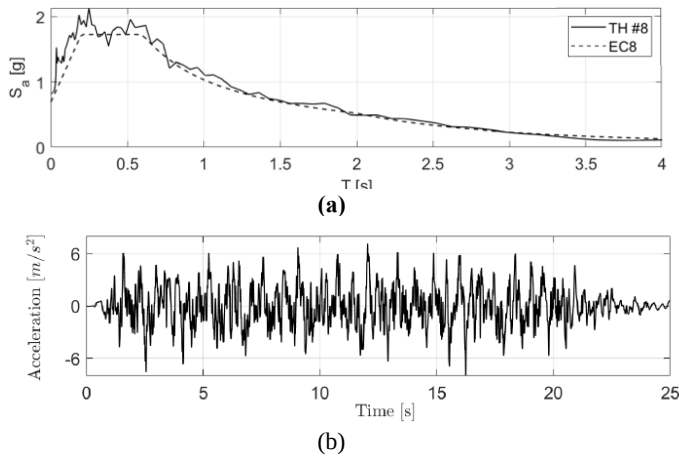


FIGURE 6: a) EUROCODE 8-PART 1 TARGET SPECTRUM AND ARTIFICIALLY GENERATED SPECTRUM FOR TH #8. ACCELERATION b) TIME HISTORIES TH #8.

In compliance with the spectrum, from a set of 10 stochastically independent accelerograms, TH #1-TH #10, TH #8 (Fig. 6b) was selected as best candidate for the experimental test campaign since the high amount of damage induced in FE model simulations.

3.2 Test programme

The testing campaign and the corresponding nomenclature is collected in Table 2. In detail, the first two runs were conducted with the random excitations RND #13 and RND #15. Successively, runs with PGA levels of 0.16 g, 0.23 g and 0.31 g were carried out. Before the test to be conducted at a level of 0.44

g, a further tuning of the shaking table was performed with a random excitation RND #16 and a relevant seismic excitation at 0.23 g PGA. After the successful completion of the test with a PGA level of 0.44 g, a last tuning phase was performed with a random excitation RND #17 and a seismic excitation with 0.23 g PGA. The final tests were executed for the maximum levels of 0.64 g and 0.71 g PGA. Lastly, a closing test with the random excitation RND #18 was carried out.

4. DYNAMIC IDENTIFICATION

Within the framework of SPIF, two approaches have been followed: i) the adoption of the power spectral density (PSD) of recorded accelerometers' signals, to provide robust indicators and rapid checks on the structural behaviour of the mock-up along the experimental campaign; ii) the use of rational fraction polynomials (RFP) applied to frequency response functions (FRFs) in order to adopt a more reliable technique for SI of frequencies, mode shapes and, especially, damping ratios. Results of SI either along the MRF direction, i.e. the shaking direction, and the orthogonal BF direction are reported in Tab. 3.

TABLE 3: IDENTIFIED FREQUENCIES FROM RANDOM SIGNALS THROUGH PSDS ANALYSES BOTH IN MRF AND BF DIRECTIONS.

	MRF Direction			
	$f_{n1}[Hz]$	$f_{n2}[Hz]$	$f_{n3}[Hz]$	$f_{n4}[Hz]$
Random #13	2.91	5.20	6.33	8.73
Random #15	2.87	5.17	6.41	8.87
Random #16	2.88	5.13	6.36	8.82
Random #17	2.86	4.99	6.33	8.62
Random #18	2.84	4.68	6.38	8.57

	BF Direction			
	$f_{n1}[Hz]$	$f_{n2}[Hz]$	$f_{n3}[Hz]$	$f_{n4}[Hz]$
Random #13	5.21	7.12	-	14.48
Random #15	5.01	7.10	-	14.45
Random #16	4.94	7.13	-	14.38
Random #17	4.95	7.14	10.12	13.52
Random #18	4.75	6.38	9.69	13.30

The response of the structure along the MRF direction, subjected to random inputs of PGA intensity in the range 0.05g – 0.15g, allows to reliably estimate the first frequency around 2.9 Hz and other frequencies at 6.0 Hz, 9.0 Hz as other main frequencies. There is also a peak around 5.0 Hz, especially pronounced with FFT's signals recorded at the 3rd floor. Instead, for the same random excitation along the BF direction, peaks near 5.2 Hz and 7.2 Hz have been identified.

As mentioned above, data have also been processed by means of the RFP, [9]. This technique overcomes many of the numerical analysis problems associated to parameter estimation of structures, such as damping ratio estimation, high computational burden for processing, etc.

Firstly, we compute the matrix of frequency response functions (FRFs) obtained from excitation signals of the shaking table and the corresponding response signals of the whole structure. Then, in order to define system's transfer function we perform the ratios between input and outputs in the frequency domain.

TABLE 2: NOMENCLATURE, CHARACTERISTICS AND SYNTHETIC OBSERVATIONS OF SPIF TEST PROGRAMME.

Label	Excitation	State	Scaling factor	PGA Level	Observations
RND #13	Random	OBE - OP	-	0.05 g - 0.15 g	- Overall elastic linear behaviour
RND #15	Random	OBE - OP	-	0.05 g - 0.15 g	- Overall elastic linear behaviour
TH #8-25%	Seismic	OBE - OP	25%	0.16 g	- Overall elastic linear behaviour
TH #8-37%	Seismic	OBE - OP	37%	0.23 g	- Overall elastic linear behaviour
TH #8-50%	Seismic	OBE - DL	50%	0.31 g	- Overall elastic linear behaviour
RND #16	Random	OBE - OP	-	0.05 g - 0.15 g	- Overall elastic linear behaviour
TH #8-37%	Seismic	OBE - OP	37%	0.23 g	- 3 rd floor: collapse of pipe rack in the transversal direction
TH #8-70%	Seismic	OBE - DL	70%	0.44 g	- Large displ.($\pm 30\text{ mm}$) of vertical Tank #2 at the 1 st floor
					- Relevant rotation of the cross beam under the vertical Tank #2 and warping of the web of fin plate D-FP #1
					- Strengthening of the cross-beam underneath Tank #2 in corresponsance of D-FP #1
RND #17	Random	OBE - OP	-	0.05 g - 0.15 g	- Overall elastic linear behaviour
TH #8-37%	Seismic	OBE - OP	37%	0.23 g	- Overall elastic linear behaviour
TH #8-100%	Seismic	SSE - SD	100%	0.64 g	- D-FP #2,5,6 fin plate experienced cracking
TH #8-110%	Seismic	SSE - NC	110%	0.71 g	- Cracking on previously detail D-FP #2,5,6 significantly increased
					- Failure of bracing supporting the conveyor
RND #18	Random	OBE - OP	-	0.05 g - 0.15 g	- Overall elastic linear behaviour

Therefore, we assume that the response to the input applied at location q is measured at location p , and thus, the related frequency response function (FRF) can be expressed as follows,

$$H_{pq}(\omega) = \frac{Y_p(\omega)}{F_q(\omega)} = \sum_{k=0}^n \beta_k (i\omega)^k / \sum_{h=0}^m \alpha_h (i\omega)^h \quad (1)$$

Finally, by curve fitting the analytical form in Eq. (1) to measurement data, the unknown coefficients $(\alpha_h; h = 0, \dots, m)$ and $(\beta_k; k = 0, \dots, n)$ of both the numerator and denominator polynomials can be determined in a least-squared error sense.

More precisely, the poles $\lambda_r = \sigma_r + i\omega_{d,r}$, that hold the information about damped frequencies $f_{d,r} = \omega_{d,r}/2\pi$, are obtained as roots of the denominator polynomial; the mode shapes instead are provided by the numerator. Moreover, damping ratios are calculated as $\lambda_r = \text{Re}(\lambda_r)/\lambda_r$, where r denotes the relevant mode shape number.

For the sake of brevity, Fig. 7 summarizes the main results in terms of eigenvalues -frequencies, damping ratios and relevant mode shapes.

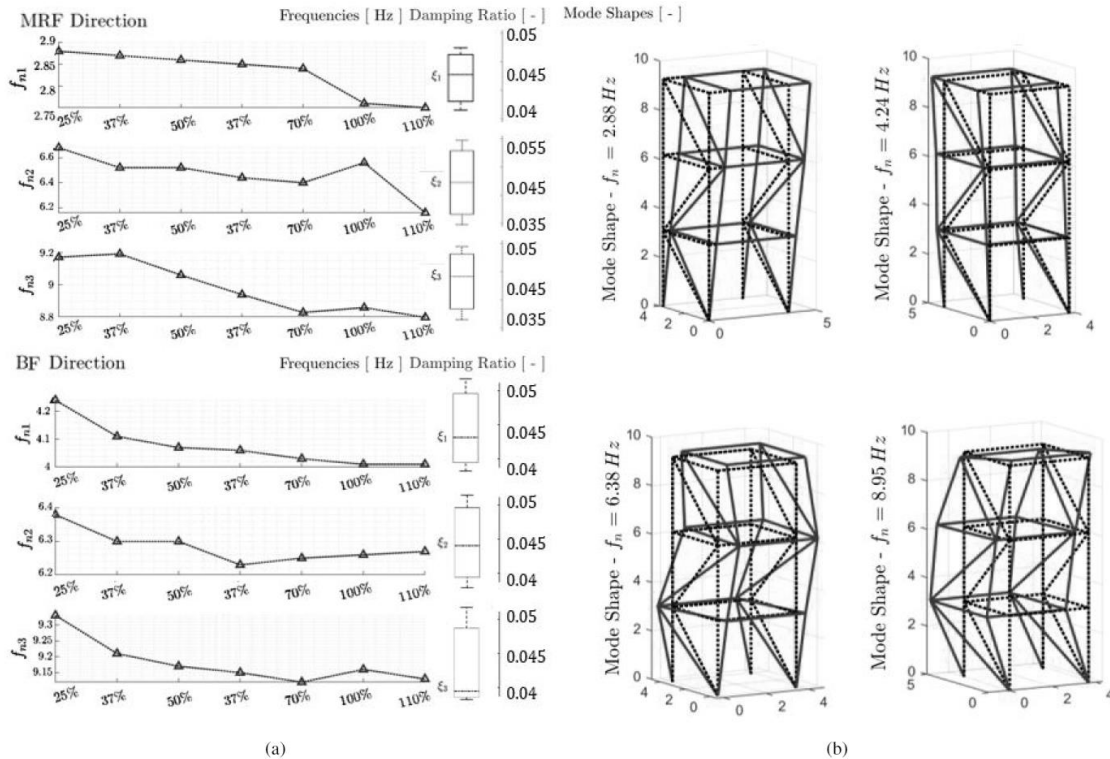


FIGURE 7: a) FREQUENCY VARIATIONS FOR BOTH THE MRF AND THE BF DIRECTION RELEVANT TO THE WHOLE EXPERIMENTAL PROGRAMME. DAMPING RATIO BOX-PLOTS ARE REPORTED WITH WHISKERS REFERRING TO THE 10th AND 90th-QUANTILE; b) MODE SHAPES RELEVANT TO FREQUENCIES $f_n = 2.88 - 4.24 - 6.38 - 8.95\text{ Hz}$.

5. SEISMIC RESPONSE AND DATA ANALYSIS

5.1 System performance

Main observations and comments on the structural response of both primary systems and process units subjected to the test programme of Table 2 are summarized herein.

The main structure and the process components behaved elastically. The elastic regime is confirmed by sensors and visual inspection of the mock-up that resulted in no damage detection both in the primary structure and secondary elements.

However, the vertical Tank #2 located in the first floor and depicted in Fig. 1 was of particular interest, due to the considerable rocking. Moreover, it is worth noticing the collapse of one suspended pipe system at the third floor.

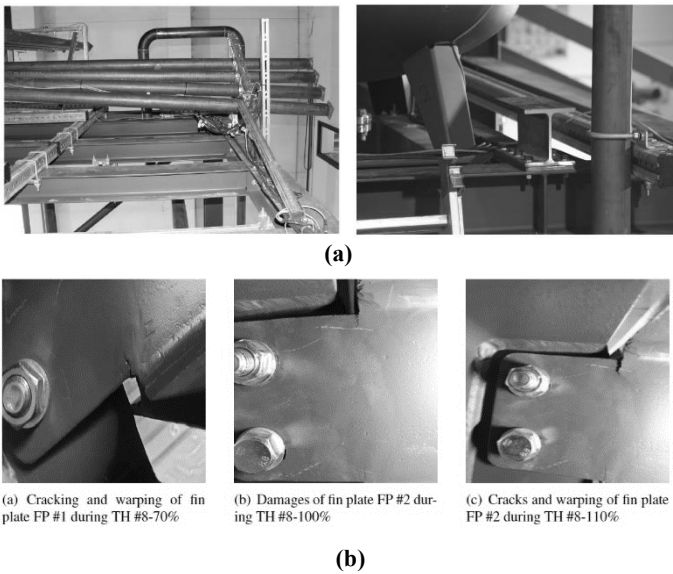


FIGURE 8: (a) COLLAPSE OF THE PIPE RACK AT THE THIRD FLOOR AND ADDITIONAL BOLTED MEMBER; (b) CRACKS AND WARPING OF FIN PLATE CONNECTIONS

During the test with TH #8-70%, large displacements of Tank #2 of about 25 mm caused additional rocking on the supporting crossbeams. As a consequence, the interaction between the main beams of the MRF frame and the crossbeams caused damage to the fin plate joint FP #1.

More precisely, the FP #1 fin plate connection experienced warping of crossbeam's web and cracks in the transition zone from beam to web. After the run TH#8-70%, in order to avoid collapse due to the strong interaction at higher PGA levels, the crossbeam underneath Tank #2 was strengthened by a bolted tee stub as depicted in Figure 8(a). The crack initiation, propagation and the entailing warping was then more consistent in the subsequent runs with higher PGA-levels also on other crossbeams, as reported in Fig. 8(b).

As anticipated, both vertical and horizontal tanks were monitored. In this respect, both absolute acceleration and relative displacement at the top of Tank #2 - between markers #36 and #5- and Tank #4 - between markers #53 and #42 - subjected to

the record TH #8-110% are shown in Fig. 11(a) and (b).

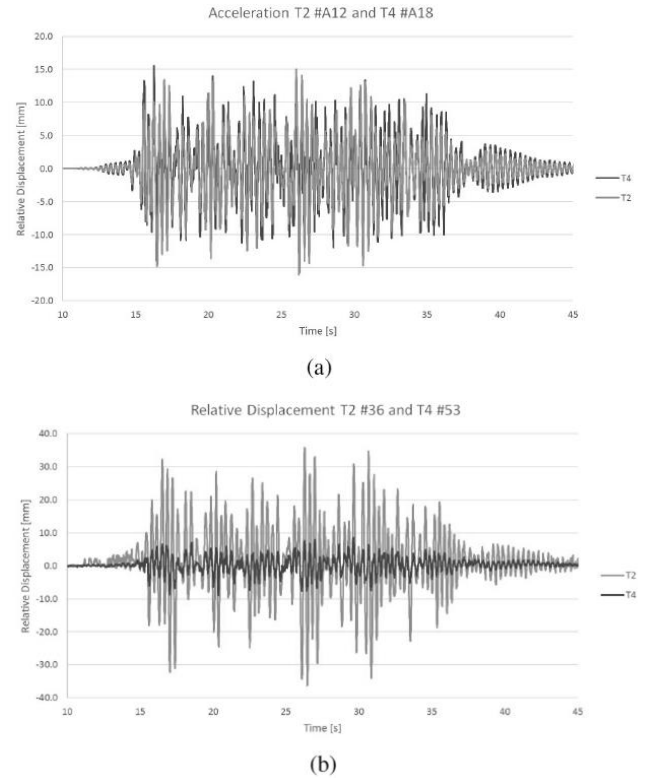


FIGURE 9: COMPARISON OF TIME HISTORIES FOR TANK #2 AND TANK #4 SUBJECTED TO THE RECORD TH #8-110%: (a) ACCELERATION; (b) RELATIVE DISPLACEMENT

5.2 Data Analysis and Insights

It is worthwhile to underline that the dynamic interaction between primary structure and process equipment cannot be ignored in seismic analysis when designing process or secondary elements.

In order to analyze this issue in depth, floor response spectra (FRS) at first and second floor have been directly derived by experimental data.

Floor acceleration spectra for each of the three floors and a damping ratio of $\xi = 4.5\%$ are depicted in Fig. 12, where the damping value derives from experimental data.

With regard to the amplitude, the high spectral values at the first floor, i.e. 1.5 m/s^2 denote a predominant linear behaviour of the primary structure.

Besides EN1998-1 [8] and ASCE07-10 [10], several authors proposed refined approaches for the prediction of FRS. In this respect, it is worthwhile to recall the methods proposed by Calvi and Sullivan [11] and by Vukobratovic and Fajfar [12]. In particular, the first approach is based on a dynamic amplification coefficient for secondary components as a function of the primary/secondary period ratio and the damping ratio of nonstructural components. The authors also accounted for the nonlinear behaviour of the primary structure by using an

equivalent period as a function of the kinematic ductility. The second approach instead, combines spectral accelerations of the primary and secondary system subjected to the ground acceleration by means of the well-known SRSS rule

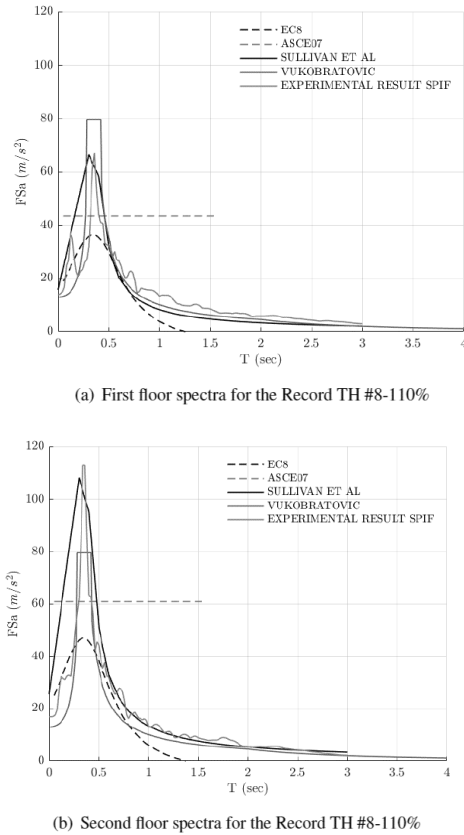


FIGURE 10: COMPARISON BETWEEN ANALYTICAL AND EXPERIMENTAL FLOOR RESPONSE SPECTRA

Moreover, this method can account for the nonlinear behaviour of the primary structure.

In addition, an importance factor $I_p = 1.5$ was used for ASCE07-10 with a behaviour factor of 2.5 and a q factor of 1.5 for EN-1998, respectively. For the latter a component amplification factor equal to 2.5 has also been assumed. Relevant results are depicted in Figure 12 and the reader can observe a general underestimation of amplitude values at resonance conditions for both EN-1998 and ASCE07-10 standards, in line with the findings of [7].

Nonetheless, one can deduce from Table 3, where both Tank #1 and #3 are considered, that the aforementioned method cannot accurately predict the FRS ordinate at the natural frequency of tanks. Differently, the method proposed by [12] appears more reliable in both the prediction of spectral shape, see Fig. 12, and the estimation of floor spectral accelerations relative to tanks, i.e. Table 3. Also in this case, a ductility value of 1.5 has been assumed to be consistent with the standard EN1998-1. A general underestimation close to the resonance condition is also observed; whereas a significant overestimation is present in other period ranges.

The main reasons behind the inaccurate estimations of both codes are related to: i) the absence of damping ratio for floor spectra estimation; ii) a general overestimation of the behaviour factor for secondary elements [11]. These considerations are reflected in the prediction of acceleration values of Table 3 where experimental floor accelerations were evaluated at the fundamental period of Tank #1 - 0.11 sec - and Tank #3 - 0.08 sec -, respectively.

6. CONCLUSIONS

The SPIF project -Seismic Performance of Multi-Component Systems in Special Risk Industrial Facilities- was proposed within the framework of the European H2020 - SERA (Seismology and Earthquake Engineering Research Infrastructure Alliance for Europe) funding scheme -. It focuses on the investigation of the seismic behavior of a representative industrial moment resisting frame substructure equipped with process secondary components by means of shaking table tests.

TABLE 3: COMPARISON OF FLOOR RESPONSE SPECTRA ORDINATES AT THE FUNDAMENTAL PERIOD OF TANKS #1 AND #3

PGA (g)	Tank #	Experimental acceleration (m/s^2)	FRS acceleration (m/s^2)	EN 1998-1 acceleration (m/s^2)	ASCE-07 acceleration (m/s^2)	Sullivan et al. (m/s^2)	Vukobratovic and Fajfar (m/s^2)
0.31	1	7.68	10.96	10	20	22	7
	3	12.82	14.31	27	27	40	13
0.71	1	15.06	30.10	22	43	32	14
	3	18.48	25.10	25	60	53	17

The testing campaign demonstrated a dynamic interaction between the primary steel structure and secondary process units that clearly influenced the performance of the whole system. More precisely, the primary steel structure remained undamaged also for SSE earthquake levels mainly due to its design and inherent flexibility that avoided a strong interaction with secondary components.

Some damage instead, appeared in web plates of supported crossbeams in the area of fin plate connections between main members and secondary beams; that happened due to the interaction between primary and secondary components, that justified a light seismic retrofit intervention in that area by means of a bolted tee stub. Moreover, results clearly showed a general overconservatism of current design standards with respect to floor response spectra prediction of process units evaluated at their fundamental period, due to the lack of consideration of damping and overestimation of behaviour/reduction factors.

Overall, it was shown that a proper experimental test campaign can highlight complex interactions and potential damage locations present in industrial plant substructures. The efficiency of other steel configurations, e.g. a braced frame configuration, also in view of proper seismic retrofit interventions deserves further studies.

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MDOF structures., *Earthquake Engineering Structural Dynamic* 45 (2016) 2495–511.

REFERENCES

- [1] E. Krausmann, E. Renni, M. Campedel, V. Cozzani, Industrial accidents triggered by earthquakes, floods and lightning: Lessons learned from a database analysis, *Natural Hazards* 59 (1) (2011) 285–300. doi:10.1007/s11069-011-9754-3.
- [2] M. Campedel, Analysis of major industrial accidents triggered by natural events reported in the principal available chemical accident databases, Tech. rep., JRC42281, Joint Research Centre Institute for the Protection and Security of the Citizen, Ispra, Italy, report EUR 23391 EN – 2008 (2008).
- [3] STREST, 2016. Harmonized approach to stress tests for critical infrastructures against natural hazards. In: Tsionis, G., Pinto, A., Giardini, D., Mignan, A. (Eds.), Report on lessons learned from recent catastrophic events, <http://dx.doi.org/10.2788/17066>.
- [4] C. Butenweg, al. et., Seismic performance of multi-component systems in special risk industrial facilities, Tech. rep., Deliverable D10.1, SERA Project, Project. No: 730900, H2020-EU – Seismology and Earthquake Engineering Research Infrastructure Alliance for Europe (2020).
- [5] V. Vukobratović, P. Fajfar, A method for the direct estimation of floor acceleration spectra for elastic and inelastic MDOF structures., *Earthquake Engineering Structural Dynamic* 45 (2016) 2495–511.
- [6] C. J. Petrone, G. Magliuolo, G. Manfredi, Floor response spectra in RC frame structures designed according to Eurocode 8, *Bulletin Earthquake Engineer* (2016) 14:747–767 DOI 10.1007/s10518-015-9846-7 (2015).
- [7] P. Calvi, T. J. Sullivan, R. Nascimbene, Towards improved floor spectra estimates for seismic design., *Earthquake and Structures* 4 (2013) 109–132.
- [8] European Committee for Standardization, Design of structures for earthquake resistance. Eurocode 8-1, CEN/TC 250, Brussels, (2004).
- [9] M. H. Richardson, D. L. Formenti, Parameter Estimation From Frequency Response Measurements Using Rational Fraction Polynomials., *Proceedings of the International Modal Analysis Conference & Exhibit* (1982) 167–181.
- [10] ASCE/SEI, Minimum Design Loads for Buildings and Other Structures, Standard 7-10, 2010.
- [11] P. Calvi, T. J. Sullivan, Estimating floor spectra in multiple degree of freedom systems., *Earthquake and Structures* 7 (2014) 17–38. doi:10.12989/eas.2014.7.1.017.
- [12] V. Vukobratović, P. Fajfar, A method for the direct estimation of floor acceleration spectra for elastic and inelastic